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National Cooperative Highway Research Program

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NCHRP Synthesis 266

Dynamic Impact Factors for Bridges

A Synthesis of Highway Practice

Transportation Research Board
National Research Council

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Synthesis of Highway Practice 266

Dynamic Impact Factors for Bridges

DAVID I. McLEAN, Ph.D, P.E.
Washington State University
and
M. LEE MARSH, Ph.D., P.E.
BERGER/ABAM Engineers, Inc.

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to state DOT and consulting bridge, structural, and research engineers. The synthesis describes the current state of the practice for determining dynamic impact factors for bridges. Information for the synthesis was collected by surveying U.S. and Canadian transportation agencies and by conducting a literature search using domestic and foreign sources.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board documents relevant background and recent information with regard to vehicular dynamic load effects on bridges. It provides details on the basic concepts of bridge dynamics, including identification of the main variables affecting bridge dynamic response. In addition, current code provisions for accounting for vehicular dynamic load effects for new bridge design and load evaluation of existing bridges are reported, including a discussion on the background of the provisions. Finally, a discussion of observed field problems associated with vehicular dynamic load effects, as obtained from the survey, are included.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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ACKNOWLEDGMENTS

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This study was managed by Stephen F. Maher, P.E., Senior Program Officer, who worked with the consultant, the Topic Panel, and the Project 20-5 Committee in the development and review of the report. Assistance in Topic Panel selection and project scope development was provided by Sally D. Liff, Senior Program Officer. Linda S. Mason was responsible for editing and production.

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DYNAMIC IMPACT FACTORS FOR BRIDGES

SUMMARY

The dynamic response of a bridge to a crossing vehicle is a complex problem affected by the dynamic characteristics of both the bridge and the vehicle and by the bridge surface conditions. Many of the parameters interact with one another, further complicating the issue, and consequently, many research studies have reported seemingly conflicting conclusions. As a result, there is considerable variation in the treatment of dynamic load effects by bridge design codes in different countries.

The provisions in the AASHTO *Standard Specifications for Highway Bridges* (1996) specify dynamic load effects in terms of an impact factor that is simple in expression, empirically derived based on railway experience, and solely a function of bridge span. These AASHTO provisions seem to have served well for many years. However, research has demonstrated that bridge dynamic response is significantly affected by a number of parameters other than span, including bridge fundamental frequency, roadway roughness approach condition, bridge type and geometry, vehicle weight and number of axles, and number of vehicles. As a result, questions have been raised in regard to the appropriateness of the continued use of the AASHTO impact provisions.

Many new bridge design codes include provisions for a dynamic load allowance to account for all vehicular load effects, not just impact. The 1994 AASHTO LRFD Bridge Design Specifications specifies dynamic load allowances that are constant; other codes include the effects of variables such as bridge fundamental frequency and number of axles. However, survey responses obtained for this synthesis indicated increased complexity to the designer as a result of including frequency calculations in determining dynamic load allowances.

Responses to a survey of U.S. and Canadian transportation agencies conducted for this synthesis indicate that 98 percent of the U.S. agencies are using the AASHTO *Standard Specifications* for the design of new bridges. Only one agency is exclusively using the *LRFD Specifications.* All Canadian agencies responding to the survey are using the CAN/CSA-S6 *Design of Highway Bridges* (1998). For overload permits, approximately half of the agencies responding reduce or eliminate dynamic load effects provided vehicle speed is restricted.

Approximately 75 percent of the agencies responding have experienced possible problems in existing bridges attributable to dynamic load effects from vehicles. Common observed problems were expansion joint failures and fatigue cracking in the girders, connections, bearings, and concrete decks of steel bridges. Improved expansion joint details, increased inspection, improved weld details, and the use of bolted connections have been used where possible to mitigate the observed problems. There were no reports of significant structural damage as a result of vehicular dynamic loading.

Research into the specific causes of the observed damage in the steel bridges is suggested. It is also suggested that procedures for accounting for dynamic load effects be as simple as possible Complicated provisions may only marginally improve the ability to properly account for vehicular dynamic load effects, and they may in fact give a false sense of accuracy.

CHAPTER ONE

INTRODUCTION

The dynamic effects produced by vehicles crossing a bridge are much more difficult to precisely quantify than are the static effects. However, most design specifications have historically prescribed simplified methods of calculating the dynamic effects so that the design process is kept simple yet presumably adequate. It is the objective of this synthesis to present and discuss the methods and available data used to quantify and understand the dynamic effects of vehicle loadings on bridges.

BACKGROUND

Loads associated with a vehicle crossing a bridge consist of the live loading resulting from the weight of the vehicle and the dynamic forces due to oscillations of the vehicle on its suspension system as well as those forces induced by the dynamic response of the bridge. For design, both the vehicle weight and the dynamic effects must be treated in a consistent manner that accounts for the variability and uncertainty in the forces and which includes an appropriate factor of safety against an overload condition occurring. The live loading can be reasonably quantified based on the static weights of actual and design vehicles. In comparison, determining the precise dynamic forces induced by vehicles is complex, somewhat abstract, and difficult to quantify.

Most bridge design codes typically specify the dynamic loading from vehicles as a fraction of the design live loading that is added to the static load. Traditionally, this dynamic loading fraction has been referred to as the "impact factor," although the term "dynamic load allowance" is considered more descriptive and encompassing and thus is becoming more popular. Failure to properly account for dynamic loading can lead to excessive bridge stresses that may cause failure in parts or in all of the bridge. Dynamic loading must also be considered when estimating the stress cycles that contribute to fatigue in the bridge components.

In the current AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996), dynamic loading is expressed as an impact factor that is a function solely of the span of the bridge. The AASHTO impact factor is empirically derived and originated based on experience with dynamic forces generated by steam locomotives on railway bridges in the early part of the twentieth century. The current AASHTO equation for impact has remained essentially unchanged since the 1920s, and it is a simple equation that attempts to account for the complex dynamic response of a bridge due to a crossing vehicle. The current AASHTO impact provisions seem to have served very well for many years, with apparently little or no significant structural distress resulting from their use. However, modern bridge designs utilize lighter materials and longer spans, and some questions have been raised regarding the appropriateness of the AASHTO impact provisions.

Research over the last 30 years has shown that the dynamic response of bridges under vehicular loading is influenced by many parameters other than bridge span. In particular, the dynamic characteristics of the vehicle, the dynamic characteristics of the bridge, and variations in the surface conditions of the bridge and approach roadways have all been shown to have a strong influence on the dynamic loading. As a result of this research, procedures significantly different from those prescribed by the AASHTO provisions have been adopted by a number of organizations around the world. For example, the Canadian code (CAN/CSA-S6, 1988) and the Swiss code (SIA 160, 1988) for the design of highway bridges both specify dynamic load allowances that are functions of the fundamental frequency of vibration of the bridge. Additionally, the dynamic loading provisions in several of the other codes are more conservative than the AASHTO provisions.

PURPOSE AND SCOPE OF SYNTHESIS

The purpose of this synthesis is to document relevant background and recent information in regard to vehicular dynamic load effects on bridges. The basic concepts of bridge dynamics are reviewed, including identification of the main variables affecting bridge dynamic response. A review of domestic and foreign literature on bridge dynamic loading was performed, and the main findings and conclusions reached from the various studies are summarized. (There are several instances in the text where the source documents referenced provide units of measure in U.S. Customary units. SI equivalents have been provided in most instances, except for some equations that contain constant values for which a simple conversion is inappropriate.) Current code provisions for accounting for vehicular dynamic load effects for new bridge design and load evaluation of existing bridges are reviewed, including discussion on the background of the provisions. As part of this synthesis study, a questionnaire was sent to transportation agencies in the United States and Canada to obtain information on current design and evaluation practices. A summary of the responses is included in this synthesis. Reports of observed field problems associated with vehicular dynamic load effects were also obtained with the questionnaire and are also included. Finally, key findings and conclusions are summarized, and suggestions for future research are made.

SYNTHESIS ORGANIZATION

Chapter 2 of this synthesis provides background material on the basic concepts of bridge dynamic behavior. Chapter 3 summarizes the results from previous research on the dynamic response of bridges due to vehicular loading. Included in this chapter is a discussion of the major parameters affecting bridge response. Chapter 4 presents on overview of design provisions from various bridge design codes for vehicular dynamic load effects. Results from the survey conducted for this synthesis on current design and evaluation practices and reports of field problems in existing bridges due to vehicular dynamic load effects are summarized in chapter 5. Finally, chapter 6 provides a summary of the main findings from this synthesis study along with recommendations for possible future research topics.

CHAPTER TWO

BASIC CONCEPTS OF BRIDGE DYNAMIC BEHAVIOR

FUNDAMENTALS OF DYNAMICS

The phenomenon of dynamic structural response is one that requires fairly complex mathematical treatment to adequately characterize the motion and forces within the structure. However, for the purposes of this discussion, differential calculus mathematics will be forgone. The discussion will instead be a conceptual treatment using a common format, one that describes dynamic response as a fraction or multiple of the response that would be obtained if the same forces or loads were applied statically. This is, in fact, the same approach that most design specifications use to calculate the effects of dynamic live loading. When using such an approach, the amount of response above the static value is typically called the dynamic increment and is found by multiplying the static value by an "impact fraction." Alternatively, the total response can be expressed as a multiple of the static value using an impact factor. The terms impact fraction and impact factor are often used interchangeably to describe vehicular dynamic effects. Both terms are found in research literature and in design codes, and therefore the reader should be alert to the particular definition being used. Both terms are used in the AASHTO family of documents.

In recognition of the complex behavior associated with the dynamic response from vehicular loading, several authors (e.g., ASCE 1982; ASCE 1981; Csagoly and Dorton 1978; Billing and Green 1984) have observed that the term "impact" is too limited and therefore not descriptive of the actual behavior. Instead, the current trend is to replace the term "impact fraction" with "dynamic load allowance," which represents the response from all types of vehicular dynamic effects, not solely impact.

Simple System Response

Single-Degree-of-Freedom Response

Impact Loading—A logical beginning point for discussion of structural dynamics is a single-degree-of-freedom (SDOF) system, as shown in Figure 1. This system is a simplified abstraction of actual systems that develop a single type or mode of response. Such a system is characterized by a spring element with stiffness, k; a damping element with a damping coefficient, c; a mass, M; and an applied force, F.

At any point in time, the spring may store strain energy, the mass may possess kinetic energy, and the damper may be dissipating energy. Given an initial source of input energy, for example an initial displacement from which the mass is released, the system will vibrate at a characteristic period, T, or frequency, f (the inverse of the period). During such free vibration, energy is being continually changed from strain to kinetic (and vice versa), and energy is being dissipated through damping until the energy of the system reaches zero.

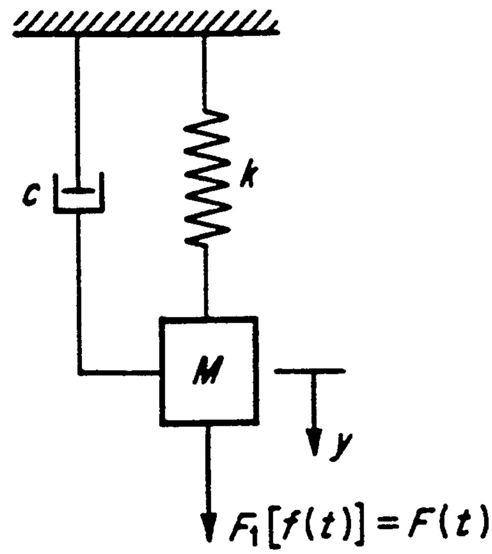

FIGURE 1 Single-degree-of-freedom (SDOF) system (from Biggs 1964).

The position of the mass cannot be changed instantaneously due to the inertial effects of the mass. Thus, when a system such as that shown in Figure 1 is loaded quickly, relative to the system period, the response is not immediately equal to the static deflection. Instead, the response takes a finite time to develop, as seen in Figure 2, where a triangular loading with respect to time is applied to an SDOF system with no damping. The response that is developed, represented by "DLF", the dynamic load factor (ratio of actual response to the maximum static response), depends on the duration of loading relative to the period of the system. Two time-histories are plotted, one for relatively slow loading and one for relatively fast loading in which the loading period, t_d, almost matches the vibration period, T. It can be seen that the duration or quickness of loading has an important effect on the maximum response that is developed. The maximum response or DLF is plotted as a function of load duration in Figure 3. It can be seen that this parameter governs whether the dynamic effect will exceed the static value (DLF = 1.0) or not. The same phenomenon is seen in bridges, although the maximum effect almost always exceeds the static effect.

Harmonic Loading—Another type of loading that is relevant to understanding the dynamic response of bridges is harmonic loading, where the load is applied as strictly a sinusoidal waveform. This is an idealization but is similar in effect to that of the more common periodic load. This type of loading is

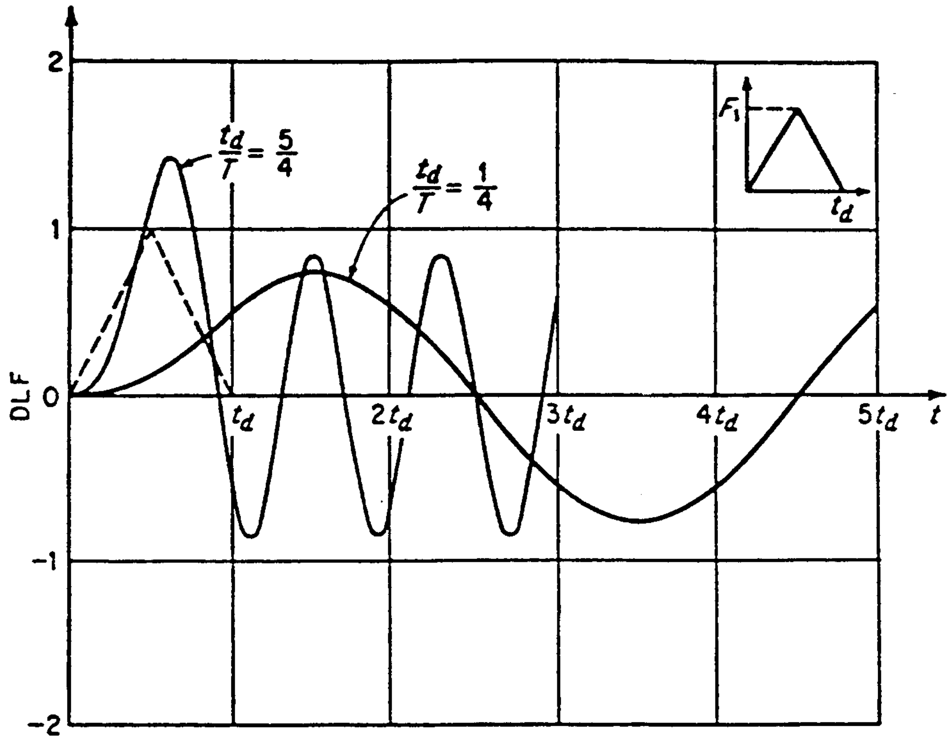

FIGURE 2 Response history of SDOF system subject to triangular pulse load (from Biggs 1964).

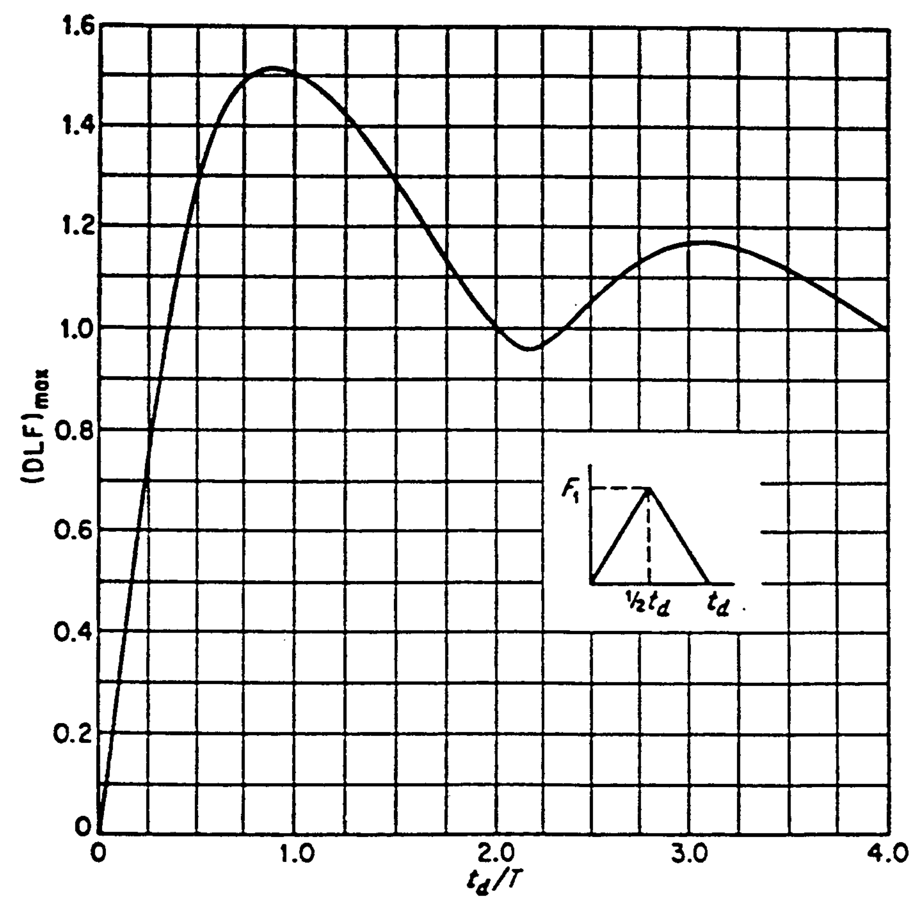

FIGURE 3 Maximum response of SDOF system subject to triangular pulse load (from Biggs 1964).

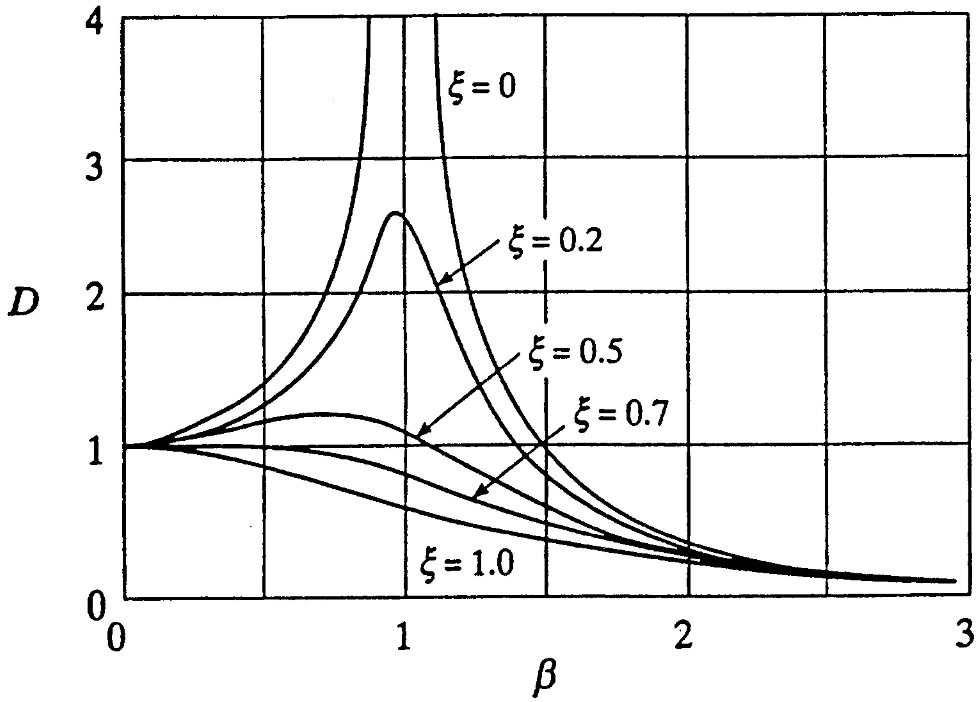

FIGURE 4 Dynamic magnification factor for harmonic loading of SDOF system (from Clough and Penzien 1993).

analogous to pushing a child in swing, and due to the regular spaced impulses of loading, the response developed can be quite large. Under periodic, and specifically under harmonic loading, the response is such that energy is continually being input into the system; such a condition is known as resonance. The only effect that limits the response from becoming large enough to cause damage is the dissipation of energy through damping. Given that most structures have small damping properties, the resonance phenomenon is to be avoided.

The condition of resonance is shown in Figure 4, in which D is the dynamic load allowance or magnification factor, β is the ratio of the loading frequency to the structure vibration frequency, and ξ is the damping ratio as a fraction of that which will allow only one-half cycle of vibration after initial release (known also as critical damping). It can be seen that large dynamic responses are indeed developed when the loading and structure vibration frequencies are relatively close (i.e., β is close to unity).

Vibration of Vehicle on Undulating Surface—The condition of driving a vehicle or pulling a trailer over an undulating roadway surface can be approximately idealized as an SDOF system under harmonic loading, provided the roadway varies as a sine wave. The loading is thus characterized by the roughness amplitude, roughness wavelength, and vehicle speed. Under such conditions, nearly resonant response can develop. If the shock-absorbing elements of the vehicle suspension system are in good condition, then the vehicle's damping is fairly high and the maximum response is limited and small. On the other hand, if the shock absorbers are worn, then the response is quite large, as indicated in Figure 4. This condition can be observed on vehicles with poor shock absorbers as "body bounce."

Distributed-Mass Beam Response

Concepts—A bridge is not a simple SDOF system, as has been discussed previously. In fact a bridge is primarily composed of beam-type components that have flexural stiffness distributed along their length, as well as distributed mass and damping properties. Thus, a more realistic model is necessary to characterize the dynamic response of actual bridges. Such distributed-property systems are typically used in the analytical studies of bridge dynamic behavior.

Natural Periods/Frequencies—The natural period of vibration of a simple-span beam with uniform cross section and weight (mass) is an important parameter that governs the response such a beam develops. The fundamental or first period of vibration (T) of a simply supported beam with uniform stiffness and mass is given by:

$$T = (2\,L^2)/\{\pi\,(E\,I/m)^{1/2}\} \tag{1}$$

where L is the span, E is Young's modulus, I is the moment of inertia and m is mass per unit length. The fundamental period of vibration for a continuous beam depends on the span geometry, but it is similar to that of a single simple-span beam if the span lengths are equal. This is due to the fact that adjacent spans vibrate in opposite directions—as one goes upward, the adjacent span goes downward. Thus the equation given can be used to approximate the vibration period for many types of bridges.

Bridge Response

Single Concentrated Load Moving Across Bridge

The simplest condition that might be considered analytically for dynamic load effects of bridges is a single concentrated constant force that moves from one end of a bridge to the other. Analytical studies of such response have been reported by Timoshenko (1928), Walker and Veletsos (1966), and AISI (1974). Figure 5 shows the maximum response developed for 1, 3, and 5 span bridges subject to a single constant force moving across the bridge at a constant speed. The plot is given in terms of the "crossing frequency" divided by the structure frequency, CF/SF. The terms of this ratio are defined as:

$$CF = v/L \tag{2}$$

where v is the velocity and L is the span length. Thus CF is the inverse of the time required to cross the span. The span frequency is the fundamental frequency of the span and is equal to the inverse of the period, T.

From Figure 5, it is seen that the time it takes to cross the span is important to the magnitude of response developed, with the maximum responses developed near a CF/SF value of 2, particularly for multi-span continuous structures. The reason is that if the time it takes to cross two spans is nearly equal to the fundamental vibration period of the structure, then a near-resonant condition develops. As the load transverses the first span, that span is deflecting downward and goes through one half-cycle of vibration. As the load enters the second span, it causes the first span to deflect upward, just as the first span enters into its second half-cycle of vibration. Thus the load is in phase with the vibratory motion.

Measured Response

While analytical treatment of the dynamic response of bridges provides a wealth of information, the actual measured response is necessary to make the connection between analysis and the "real world." Researchers have been attempting to quantify the response of bridges in the field for years, and while one cannot obtain all the information that one would like, enough has been measured to be useful. An example of a measured "time history" of response is shown in Figure 6 from Biggs and Suer (1955). This study, performed in the 1950s, measured the response of several simple-span bridges to a two-axle vehicle. The plot shows the dynamic (measured) response as a function of the position of the heavier rear axle. Also shown is the deflection measured for static response, which was obtained by driving the truck across the bridge at a

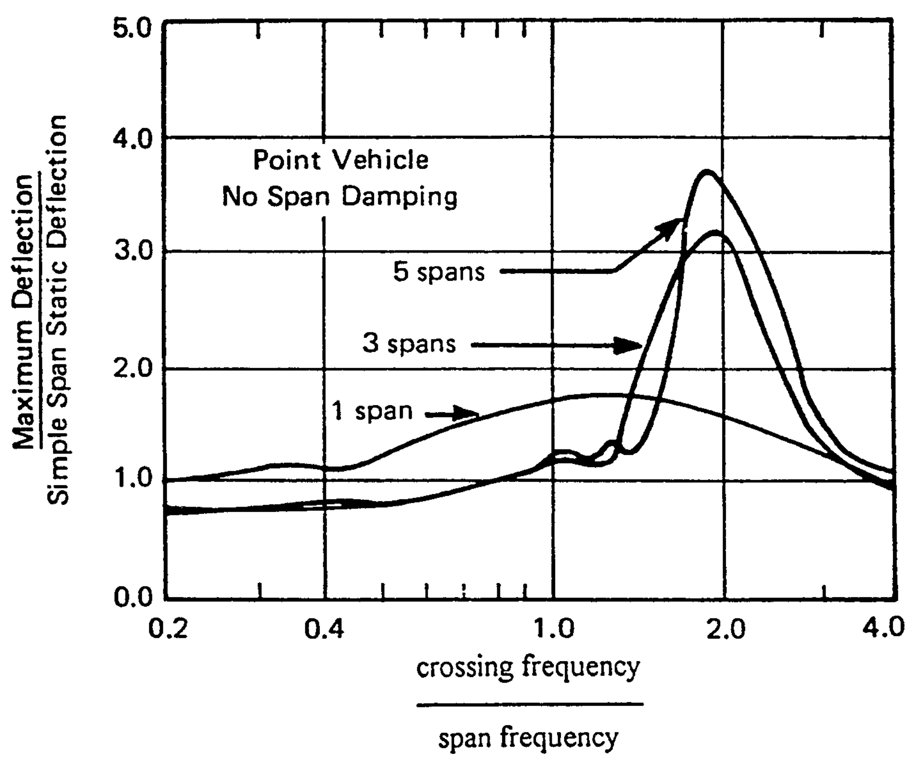

FIGURE 5 First span deflections for moving constant point load (from AISI 1974).

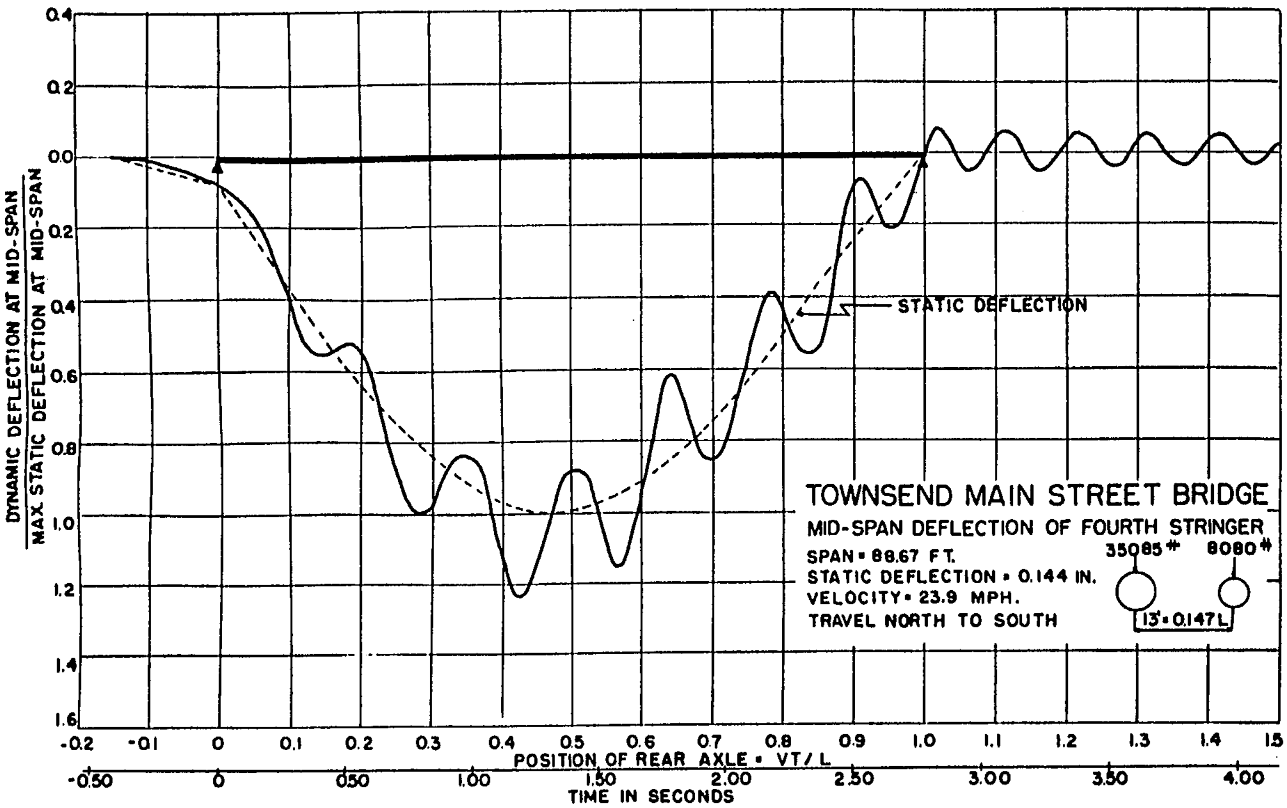

FIGURE 6 Measured deflection of simple-span bridge (v = velocity, T = crossing time, and L = span length) (from Biggs and Suer 1955).

crawl speed. The ordinate is normalized to report the dynamic response in terms of the dynamic amplification.

There are several key points to note from Figure 6. First, the maximum dynamic response does not occur at the same time as the maximum static response, nor at the location of the truck. Second, the build-up of dynamic response is evident by the increased vibration amplitudes seen in the right side of the plot. Third, the free vibration of the bridge after the vehicle has left the structure is seen at the very right of the figure. In the free vibration phase, the decrease in amplitude due to energy being dissipated by damping is seen. Often the damping values for a bridge are determined by measuring this decrease and applying the "logarithmic decrement" method (Tilly 1978; Clough and Penzien 1993; Biggs 1964).

Definitions of Dynamic Response

It is important to define just what the dynamic effect is, since there are many ways to calculate a dynamic load increment, dynamic load allowance, dynamic magnification factor, or whatever such an effect is being called. Bakht and Pinjarkar (1989) have pointed out eight different mathematical definitions that have been used for calculating dynamic load allowance (DLA) from either analytical or test data.

There are at least three common definitions for the DLA. All of the methods use the static response due to a truck "crawling" across a bridge as the reference with which to define DLA. One common method is the definition used with the data shown in Figures 3 and 6. The DLA is simply the maximum instantaneous dynamic response divided by the maximum static response developed. Many analytical investigations "normalize" the data in this manner and, as seen in Figure 6, some field data are related in this fashion. With this definition, a DLA of approximately 0.25 is obtained from the figure. As mentioned earlier, the maximum dynamic increment and the maximum static values do not occur simultaneously.

A second method is to divide the dynamic response that occurs at the same location as the maximum static response by the maximum static value. The fallacy of using this method is seen in Figure 6 where the dynamic response is nearly equal to the maximum static response when this static value occurs at about 0.48 of the span.

The third common method is to divide the maximum dynamic response by the static response that occurs simultaneously with the maximum dynamic response. In the case shown in Figure 6, such a definition would result in a slightly higher DLA than the 0.25 value indicated earlier.

The third definition is perhaps the most precise of the three given here and was apparently used to interpret many of the Ontario tests (Bakht and Pinjarkar 1989). However, for the purposes of design as currently practiced, it appears that the first definition is the most rational. This is because, in design, the maximum static effect is scaled to give the maximum dynamic effect regardless of when the two responses occur, and this is precisely what the first definition produces.

VARIABLES AFFECTING BRIDGE RESPONSE

This section briefly describes several of the variables that have been historically shown to be important in determining the maximum dynamic response of bridges to vehicular traffic. More detailed discussions on the effects of the variables are given in chapter 3.

Speed

Vehicle speed has been shown to be an important parameter in as much as a vehicle traversing a bridge at a given speed may produce near-resonant response as discussed previously. However, the near resonant phenomenon seems to develop more so when an unsprung, constant force moves across a bridge rather than a vehicle riding on a typical suspension system.

Bridge Vibration Frequency

The effect of bridge vibration frequency or inversely, period, was shown in Figure 5 for a constant force moving across the structure. As with vehicle speed, the natural frequency of vibration is a key parameter that establishes whether near-resonant response is possible. In fact, it has been demonstrated that not only are the vibration characteristics of the bridge important, but that the vehicle vibration characteristics are also important in defining the dynamic response that develops in the vehicle-bridge system.

Weight of Vehicles

The weight of the vehicle causing the dynamic response logically should be an important parameter simply due to increase in exciting force with increasing vehicle weight. However, since it is the dynamic increment that is typically sought, it is not at once obvious how weight affects the increment. As will be discussed in chapter 3, researchers have established that the dynamic increment increases with weight for single-axle loads, whereas others have observed that the dynamic increment decreases with increasing number of vehicle axles, which is related to vehicle weight.

Suspension

The importance of vehicle suspension is perhaps best observed in actual field measurements as seen in Figure 7, where deflections were measured for a simple-span bridge. For one deflection trace the vehicle had its normal suspension characteristics, in the other the springs were blocked so that the truck rode directly on the axles. The increase in response is evident for the unsprung condition.

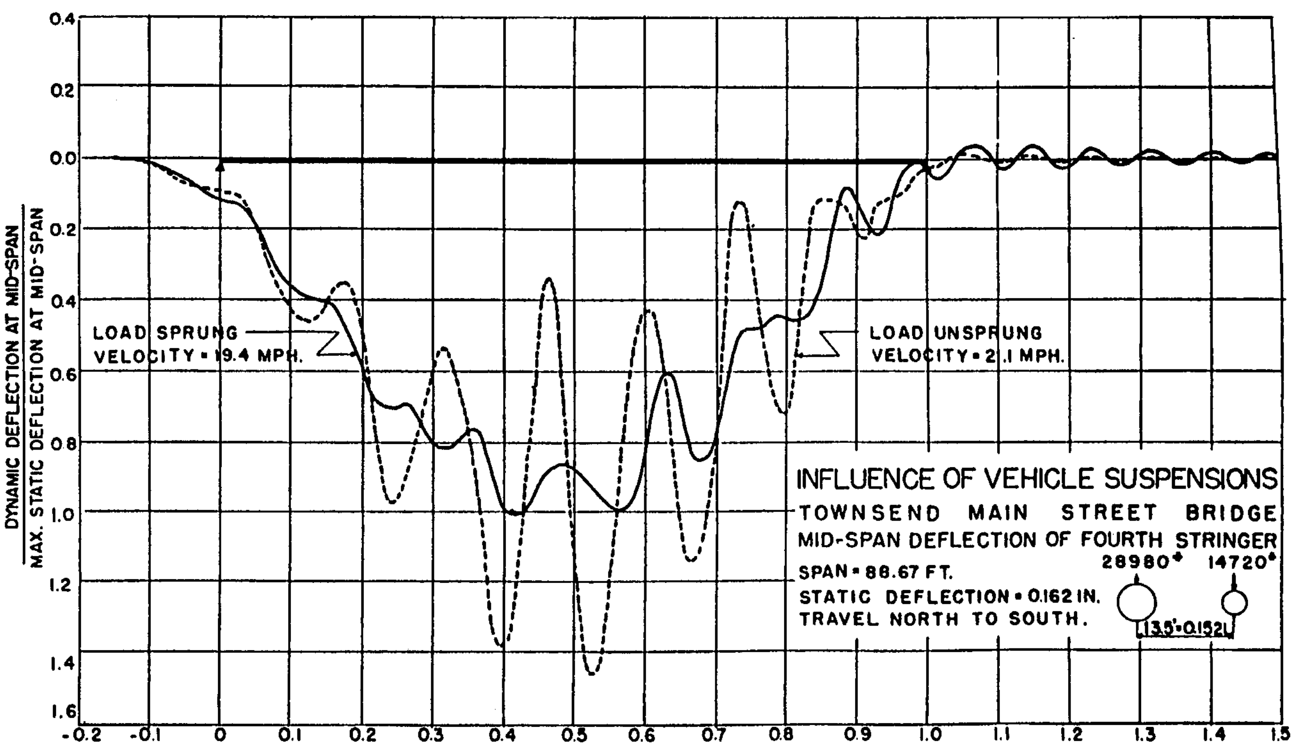

FIGURE 7 Effect of vehicle suspension on the measured response of simple-span bridge (v = velocity, T = crossing time, and L = span length) (from Biggs and Suer 1955).

Surface Roughness

The roughness of the bridge surface has been shown by many researchers to be important. Categories of roughness vary from simple pavement undulations to planks that are laid across vehicle travel paths. The response from crossing the planks is typically larger than that of simply undulating pavement, and the effects of planks may be similar to spots of missing or deteriorated overlays.

The effect of roughness on response may be visualized by considering a vehicle that is set into vibration by transversing undulating or rough pavement. As the vibrating vehicle crosses a bridge, the loading effect seen by the bridge is nearly periodic. If the period of vehicle vibration, and therefore the period of loading, is nearly equal to that of the bridge, then a quasi-resonant condition exists and the bridge may develop a relatively large response. The effect has been observed in actual bridges and will be discussed in more detail in chapter 3.

Damping

As described previously, damping determines how rapidly energy is dissipated from the bridge system after excitation is initiated. Most bridges inherently do not have significantly different damping values, and thus damping is not typically a parameter that can be adjusted to control the response of bridges. The exception to this statement occurs when damping elements are intentionally incorporated into a structure to control vibrations. While this strategy is increasingly being used for buildings, the use of dampers in bridges is still in a developmental stage. However, one reason for determining the damping in bridges is to calibrate analytical models for which accurate damping values are necessary if accurate dynamic responses are to be determined.

CHAPTER THREE

RESEARCH ON THE DYNAMIC RESPONSE OF BRIDGES UNDER VEHICLE LOADING

INTRODUCTION

The dynamic response of bridges due to vehicular loading has been a subject of interest to engineers for more than 100 years. Over the last 40 years, a significant amount of research has been conducted in the area of bridge dynamics, and this research has been both analytical and experimental in scope. The dynamic response of bridges is complicated by a number of independent but interacting factors and, as a consequence, many of the studies have produced seemingly conflicting results and conclusions. Correspondingly, there is considerable variation in the treatment of dynamic loads by bridge design codes in different countries.

This chapter presents a historical overview of the research that has been performed to investigate various aspects of the dynamic behavior of bridges due to vehicular loading. This is followed by a presentation of the main observations and conclusions in regard to the effects of relevant parameters on bridge response.

OVERVIEW OF PAST RESEARCH

Following the collapse of several railway bridges in Great Britain, the first study of vehicle-bridge interaction was conducted in 1849 by Willis (Paultre et al. 1992). Willis conducted laboratory tests of a moving load on cast iron beams and formulated the differential equation of motion for a point mass moving at constant speed over a massless but flexible beam.

Early on, the focus of most studies of bridge-vehicle interaction was on railway bridges, where dynamic forces were generated due to the hammer-blow effect of steam locomotive drive wheels. This hammer-blow effect produces a nearly harmonic force with a frequency proportional to the locomotive speed and results in relatively large impactive forces (Billing and Green 1984). In 1911, the American Railway Engineering Association (AREA) (Dhar et al. 1978) initiated a series of tests on railroad bridges that led to the following design provision for impact forces in railroad bridges:

$$I = 50/(L + 150) \tag{3}$$

where I is the impact factor, not to exceed 0.30, and L is the span length in feet. In 1927, a joint committee (ASCE 1931) of AREA and the American Association of State Highway Officials (AASHO) adopted the following equation for both railroad and highway bridge impact:

$$I = 50/(L + 125) \tag{4}$$

Thus, the impact design provisions in highway bridges largely originated from experience with railway bridges and steam locomotives.

In 1931, an ASCE committee (ASCE 1931) reported on a "search for available data on the subject of impact in highway bridges." Among the observations of the committee were that:

1. Stresses due to static loads and impact are important, in regard to the safety of the structure, only when they approach design values.
2. The percentage of impact increment decreases as the loads increase and, therefore, as the unit stresses increase.
3. Existing data are too meager to establish a relation between impact and span length. Approximately the same impacts were indicated for all spans.
4. Uncertainties in the distribution of the stress over the various parts of a bridge are much greater than uncertainties in impact.

The ASCE committee recommended that an impact increment of stress of 25 percent of the live load stress be used in the design of bridge floors and highway suspenders. Committee observation 3 above notwithstanding, the established reduction for increased spans used for railroad bridges was considered the best guide for highway bridges. Thus, for girders and trusses of highway bridges, the committee recommended an impact increment of stress as a fraction of the live load determined from:

$$I = 50/(L + 160) \tag{5}$$

with I not greater than 25 percent.

Beginning in the 1950s, new investigations of the dynamic performance of bridges were conducted, including those by Foster (1952), Hayes and Sbarounis (1955), Edgerton and Beecroft (1955), Biggs and Suer (1955) and Wright and Green (1959). In 1961, Fleming and Romualdi (1961) conducted an analytical investigation of the dynamic response of single-span and three-span bridges to transient loads. They reported impact factors higher than predicted by the AASHO provisions for short span bridges, but found that the impact factors were in accordance with the provisions for bridges with spans greater than 24 m (80 ft). The researchers also concluded that, for unsprung loads, the vehicle speed and potential unevenness of the bridge approach are the most important parameters influencing impact.

A major experimental investigation of bridge impact loads was initiated in 1958 by AASHO (1962). Eighteen simply-supported bridges were investigated, each with a span of 15 m (50 ft). Test vehicles consisting of two-axle trucks and three-axle tractor-trailer combinations were driven over the test bridges at varying speeds. Observations from these tests included:

1. Initial vertical oscillations of the test vehicle were present in practically all tests and introduced a large uncertainty in the dynamic response of the bridges.
2. The largest dynamic impact factor based on displacement measurements was 0.63, although only 5 percent of the measured values exceeded 0.40. The impact factors based on measured strains were somewhat lower; the largest value was 0.41 and only 5 percent of the values exceeded 0.29, which was the impact value specified by the AASHO provisions at the time.
3. Blocking the vehicle springs to eliminate the effectiveness of the suspension resulted in approximately doubling the dynamic impact factor.
4. Reasonable agreement was observed between the experimental results and analytical results based on dynamic spring-supported vehicle models.
5. The dynamic impact factor generally increased with increasing vehicle speed, which was characterized in terms of a speed parameter α:

$$\alpha = V/(2\,Lf) \tag{6}$$

where V = truck speed (ft/sec.), L = span (ft) and f = first flexural frequency of the bridge (Hz).

Based on the results of this research, an alternative equation for determining the impact factor was suggested as:

$$I = 0.15 + \alpha \tag{7}$$

The first term in the equation is an allowance for initial oscillations of the truck coming onto the bridge and the second term represents the effects from a smoothly rolling mass crossing a beam (Walker and Veletsos 1966).

Wright and Green (1963) reported on a series of experimental investigations of vehicle-bridge interaction made on 52 highway bridges in Ontario from 1956 to 1957. The bridges selected for study were known to noticeably vibrate under traffic loads and were of different types with widely varying approach and deck conditions. For each bridge, measurements of bridge stiffness, natural frequency, damping and dynamic amplification factor (DAF) under both normal traffic conditions and test vehicles were made. DAFs of up to 75 percent were observed, but most values were around 30 percent. The larger values were obtained for bridges with observed frequencies in the range of 2 to 5 Hz (see Figure 8). This frequency range encompasses the bounce frequencies typical for heavy commercial vehicles. The investigation also concluded that the DAF was strongly influenced by any irregularities in the bridge approaches or riding surface, including the presence of expansion joints.

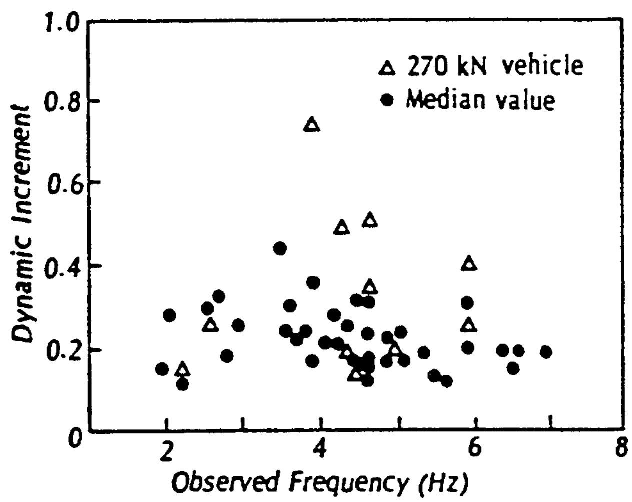

FIGURE 8 Dynamic increment versus observed frequency (from Billing and Green 1984).

Csagoly, Campbell, and Agarwal (1972) conducted a second series of tests on bridges in Ontario in 1969 to 1971. Eleven bridges were investigated. The maximum dynamic amplification factors obtained from the tests ranged from 30 to 85 percent and were again obtained for bridges with measured frequencies in the 2 to 5 Hz range. Later, the authors reanalyzed this data and compared the measurements to the provisions in the AASHTO design specifications. They concluded that the dynamic amplification factors were higher than those in AASHTO for bridges having natural frequencies in the range of 2.5 to 4.5 Hz.

In 1970, Veletsos (1970) developed a numerical method for computing the dynamic response of highway bridges to moving vehicular loads. Veletsos considered a linear elastic beam element with distributed flexibility and lumped masses. Loading was considered using a three-axle sprung mass model with appropriate damping to represent the suspension system.

Gaunt et al. (1977) investigated analytically the effects on bridge accelerations of several parameters, including the properties of the bridge and the vehicle as well as the initial conditions of the roadway. The calculated accelerations were compared to criteria for human response. Their results indicated that, for simple-span bridges, accelerations that might psychologically disturb a pedestrian are primarily influenced by bridge span, vehicle weight and speed, and most of all by roadway roughness.

Page (1976) and Leonard (1974) reported on a series of dynamic tests conducted on highway bridges by the Transport and Road Research Laboratory (TRRL) in England. The main

TABLE 1

TYPICAL VALUES OF MEASURED DAMPING IN HIGHWAY BRIDGES (from Paultre et al. 1992)

Type of Bridge	Span Length (m)	Number of Bridges Tested	Average Damping Value	Lowest Damping Measured
Concrete in Switzerland, Great Britain, and Belgium (Tilly 1986)	10–85	213	0.079	0.020
Composite, steel-concrete in Great Britain (Tilly (1986)	28–41	12	0.084	0.055
Prestressed concrete (Billing 1984)	8–42	4	0.022	0.008
Steel (Billing 1984)	4–122	14	0.013	0.004

testing program was on 30 bridges, and impact values of between 0.09 and 0.75 were reported. The researchers also reported on a laboratory study of the impact response of vehicles traveling over smoothed humps and planks. Impact values of up to 2.0 were reported. Tilly (1978) also reported on research conducted at TRRL investigating the response of bridges excited by an energy input device. From these tests, damping values for the test bridges were obtained. Damping values were found to increase with the amplitude of vibration. Typical values of damping for steel, concrete, and composite bridges are shown in Table 1 (Paultre et al. 1992) and Table 2 (Tilly 1978).

TABLE 2

TYPICAL VALUES OF DAMPING (from Tilly 1978)

Material	Component	Bridge
Steel 0.002 to 0.008	0.004 to 0.03	0.02 to 0.06
Concrete 0.01 to 0.06	0.02 to 0.06	0.02 to 0.1

Shepard and Sidwell (1973), Shepard and Aves (1973) and Wood and Shepard (1979) reported on a series of bridge tests conducted in New Zealand. Impact values were calculated from deflection measurements obtained during the passage of a standard two-axle truck over the bridges along with normal vehicular traffic. Fourteen bridges were investigated, and impact values ranging from 0.1 to 0.7 were reported. Of note, the impact value of 0.7 was obtained for a particular bridge due to normal traffic, which presumably included both light and heavy vehicles; for the same bridge, an impact value of 0.3 was obtained due to the standard test truck. The researchers noted that "vehicle characteristics have a significant impact on the recorded impact."

Sweatman (1980) of the Australian Road Research Board (ARRB) measured wheel forces in a series of test vehicles driven over a variety of road conditions. Impact was found to be a function of type of vehicle and suspension system, with the highest values reported for pivoted drive tandem systems. The ARRB also sponsored research into the impact effects associated with vehicle breaking (Gupta and Trail-Nash 1980). O'Conner and Pritchard (1985) reported on field studies carried out in 1981 and 1983 on a short span, composite steel and concrete bridge, which was instrumented to measure midspan bending moments (i.e., strains). Results from the study were for use by the ARRB in the preparation of a new bridge design code. Values of impact varied from -0.08 to 1.32 for 170 trucks in normal traffic with weight from 24 to 40 Mg (27 to 44 tons). O'Conner and Pritchard noted that the higher impact values occurred with lighter vehicles and there was a slight trend of decreasing impact values with heavier trucks. Later field measurements on the same bridge (O'Conner and Chan 1988a and 1988b) supported the same general conclusions.

In 1980, a third series of dynamic testing of highway bridges in Ontario was performed (Billing 1984; Billing and Green 1984). The tests were conducted on 27 bridges of various configurations of steel, concrete, and timber construction, and with spans of 5 to 122 m (16 to 400 ft) to obtain comprehensive data to support the *Ontario Highway Bridge Design Code* (OMTC 1979) provisions. Accelerations, deflections, and strain measurements were made to characterize the dynamic response of the bridges. More than 100 individual runs were recorded for each bridge by both test vehicles and normal traffic crossing at a variety of speeds. The approaches, expansion joints, and decks of all the test bridges were judged to be in good condition. The conclusions from this research included:

1. The mean dynamic amplifications were relatively modest, even though for some tests dynamic amplifications greater than 0.5 were observed. The coefficients of variation for the amplification magnitudes were large.
2. The mean dynamic amplifications generally decreased with increase in truck weight for bridge spans greater than 30 m.
3. When test runs were made with two vehicles side by side, the dynamic amplifications were generally reduced.
4. The largest dynamic amplifications were obtained for bridges with a fundamental flexural frequency in the range of 2 to 5 Hz.
5. Bridges of timber construction were found to apparently not vibrate.

In 1981, the ASCE Committee on Loads and Bridges (ASCE 1981) recommended that "no new provisions for highway bridges be advanced at this time" except that the term "impact" be replaced wherever appropriate by the more descriptive term "dynamic allowance for traffic loadings." The report also recommended that the AASHTO provisions be compared to dynamic load allowance specifications in the Ontario Highway Bridge Design Code (OMTC 1979) and to other national codes in effect worldwide. However, the report noted that no structural distress problems appear to have resulted from the use of the AASHTO impact allowances, and "that fact plus their simplicity would seem to mitigate against drastic changes."

Schilling (1982) reported on theoretical and experimental studies of impact factors for steel highway bridges. He reported impact factors as high as 1.0, but found that higher impact values were obtained under "unusual conditions," such as a bump at a critical location. Impact factors determined for individual trucks in traffic, or for test trucks, were generally much lower.

Cantieni (1983, 1984) presented results from the dynamic load tests of 226 highway bridges in Switzerland from the mid-1950s to the early 1980s. The research was performed for the Swiss Federal Laboratories for Materials Testing Research. Most of the bridges were prestressed concrete construction. The dynamic fraction of the total measured response was found to be as high as 0.7 for bridges with a fundamental natural frequency between 2 and 4 Hz, corresponding to the range of typical truck body bounce frequencies. The results and conclusions from these tests provided the basis for the dynamic amplification provisions in the 1988 Swiss SIA 160 code.

Inbanathan and Wieland (1987) performed an analytical investigation of the dynamic response of a simply supported box girder bridge due to a vehicle moving across the span. They considered different roughnesses by representing the roadway profile using a power spectrum of highway elevation variations and a steady-state sinusoidal relationship between dynamic force and the pavement surface elevation. Among their conclusions were:

1. The effect of vehicle mass on bridge response is more significant at high speeds.
2. Maximum response need not occur simultaneously for bending moments and deflections. It is necessary to consider both forces and deflections when evaluating dynamic response. Small variations in deflections can result in large differences in the dynamic forces.
3. Dynamic stresses developed by a heavy vehicle moving over a rough deck at high speeds were larger than those predicted by several (then) current bridge codes.

Coussy et al. (1989) presented an analytical study of the influence of random surface irregularities on the dynamic response of bridges. The vehicle spring system was represented by a spring, damper, and mass for each axle, and the vehicle was represented as a single rigid body. Random surface irregularities were represented using a stationary normal-centered random process, with the spectral density obtained from previous experimental results. Results from the investigation showed that, in the absence of significant surface irregularities, the dynamic amplification factor was independent of bridge span.

In 1989, Bakht and Pinjarkar (1989) presented a literature review of bridge dynamics, with a particular focus on the dynamic testing of highway bridges. They showed that various definitions have been used for impact factor, and as a result the same set of test data may lead to different reported estimates of impact. A number of other factors were identified as possible reasons for differences in reported dynamic amplification leading to difficulty in comparing experimental results, including vehicle type, weight, and position, type of information monitored (e.g., bridge strains vs. displacements), riding surface variations, and presence of multiple vehicles. The authors made recommendations for determining the impact factor through testing of highway bridges. Other attempts to develop standardized test procedures for the dynamic testing of bridges have been proposed by Billing and Agarwal (1990) and by Paultre et al. (1995).

Hwang and Nowak (1989, 1991) and Nowak, Hong, and Hwang (1990) developed procedures for calculating the statistical parameters of dynamic loads on highway bridges to be used in the development of a reliability-based bridge design code. Variables considered included different vehicle characteristics (mass, suspensions and tires, axle arrangement and speed), roadway roughness and bridge properties (span, mass, support type, material, and geometry). Calculations were made for steel and prestressed concrete bridges. Dynamic load factors were calculated based on the 75-year mean maximum loads. Dynamic load factors were found to be lower for heavier trucks. The factors were also lower for two side-by-side trucks compared to a single truck. Based on their results, a uniform dynamic load factor of 0.25 was recommended for all spans greater than 6 m (20 ft.)

Field measurements were conducted by Nassif and Nowak (1995) to verify the previous analytical models. Observations indicated that the dynamic load factor decreased with increasing vehicle weight and that the factors were in agreement with the analytical predictions for heavy trucks. Larger factors were obtained when calculated based on the strains in the exterior girders; however, these girders were noted to be loaded to relatively low stress levels. Therefore, the researchers recommended that dynamic amplification factors should be based on measurements from the interior girders.

Paultre et al. (1992) presented a comprehensive review of analytical and experimental findings on bridge dynamics and the evaluation of the dynamic amplification factor (DAF). The following conclusions were reached as a result of their review:

1. The DAF is related to the fundamental frequency of the bridge.
2. While the specified live loads differ in the bridge design codes that exist throughout the world, this does not justify the large differences in the treatment of dynamic load allowances in the various codes.
3. Analytical models cannot reliably evaluate the DAF for a particular bridge at the present time (circa 1992). Full-scale testing under traffic loading is the only economical and practical way to evaluate the DAF with reasonable confidence.
4. Roadway roughness, pavement irregularities and vehicle suspension systems all strongly influence the DAF. Vehicle speed and axle spacing can also be important parameters.

5. Geometry and construction materials of typical highway bridges do not seem to influence the DAF.

An extensive series of analytical investigations on the dynamic behavior of bridges was conducted by Garg, Chu, and Wang (1985), Chu, Garg, and Wang (1986), Wang, Garg, and Chu (1991), Huang and Wang (1992), Wang and Huang (1992), Wang (1990, 1993), Wang, Shahawy, and Huang (1992, 1993a, 1993b), Wang, Huang, and Shahawy (1992, 1993a, 1993b, 1994, 1996), Huang, Wang, and Shahawy (1992, 1993, 1994, 1995a, 1995b) and Wang, Huang, Shahawy, and Huang (1996). Nonlinear vehicle models were developed to represent both railway and highway vehicles. Surface roughness was represented as a random process described by a power spectral density function. Bridge models varied according to the type of bridge and application. Parameters evaluated included use (railway and highway), bridge configuration (cable stayed, thin-walled box girder, multigirder, single-span, continuous and cantilever, rigid frame), bridge layout (straight, skewed, horizontally curved), construction material (prestressed concrete, steel, reinforced concrete), vehicle models, roadway roughness (represented by the International Organization (ISO) specifications for very good, good, average, poor), ramp/bridge interface condition, vehicle weight and location, and presence of multiple vehicles. Among the general conclusions reached in this series of studies were:

1. For very good, good, and average roads, most impact factors were found to be lower than the values specified by both AASHTO and OHBD. However, very high impact factors were found for poor road surfaces.
2. The effects of vehicle speed on impact were not significant for very good, good, and average roads.
3. The impact factors in short span bridges were larger than those for long span bridges. There was a general trend of decreasing impact factors with increasing spans.
4. Impact factors decreased with increasing vehicle weight. The shorter the span, the more rapidly the impact factor increased with lessening vehicle weight.
5. The presence of damping decreased the response of a bridge, but the influence of damping on the bridge components varied.

Heywood (1995) presented the results of an investigation in Australia on the influence of truck suspensions on the dynamic response of short span bridges. Three bridges were instrumented and their dynamic response to air-suspended or steel leaf-spring-suspended test vehicles were measured. Bridge response was found to be sensitive to the natural frequency of the bridge, the suspension of the vehicle, vehicle speed, and surface roughness. Dynamic response increments of up to 1.0 were recorded, with generally a lesser response for the air-suspended vehicles unless axle hop was excited.

The dynamic response of timber bridges was investigated for railway applications by Uppal and Rizkalla (1988) and for highway applications by Ritter et al. (1995). For both studies, experimental measurements were made of the timber bridges under the passage of test vehicles. Dynamic amplification factors of up to 0.5 were reported, with the larger values occurring in the presence of track or roadway irregularities.

PARAMETERS AFFECTING THE DYNAMIC RESPONSE OF BRIDGES

The dynamic response of a bridge is the result of the modes of vibration of the bridge responding to the forcing function generated by a vehicle oscillating on its suspension system. A complete description of the dynamic response should include the mass distribution within the system, natural frequencies and modes of vibration of both the bridge and vehicle, damping characteristics of both the bridge and vehicle, initial conditions for both the bridge and vehicle, and riding surface profile (Billing and Green 1984). In practice, many of the items noted are difficult to measure and quantify. Most of the parameters interact with each other, complicating the problem further. Based on the review of past research, the effects of various parameters on the dynamic response of bridges to vehicular loading are discussed in the following section.

Bridge Parameters

Bridge Fundamental Frequency

The fundamental frequency of vibration for a bridge due to vertical flexural loading can potentially have a significant effect on the dynamic response. Field measurements of the bridge response to dynamic loads have reported that a majority of the fundamental frequencies for typical bridges are in the range of 2 to 5 Hz (see Figure 9). In general, good agreement

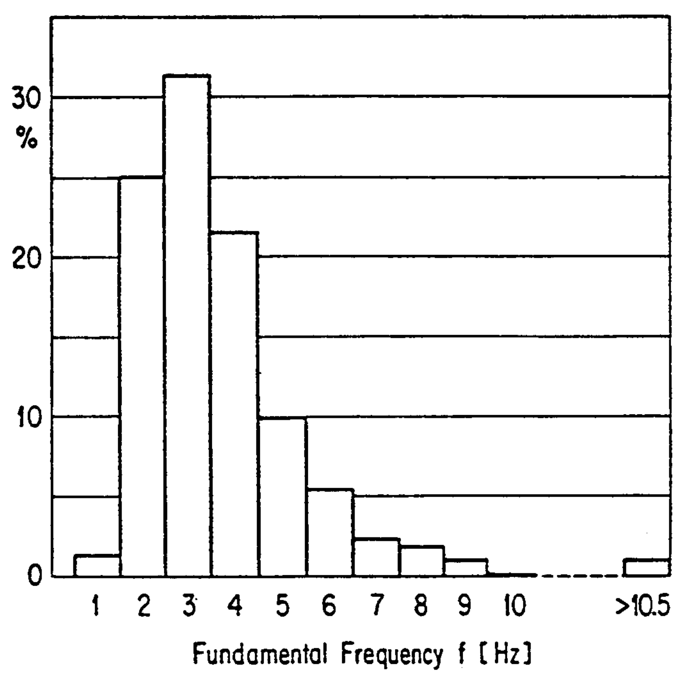

FIGURE 9 Distribution of fundamental bridge frequencies (from Cantieni 1984).

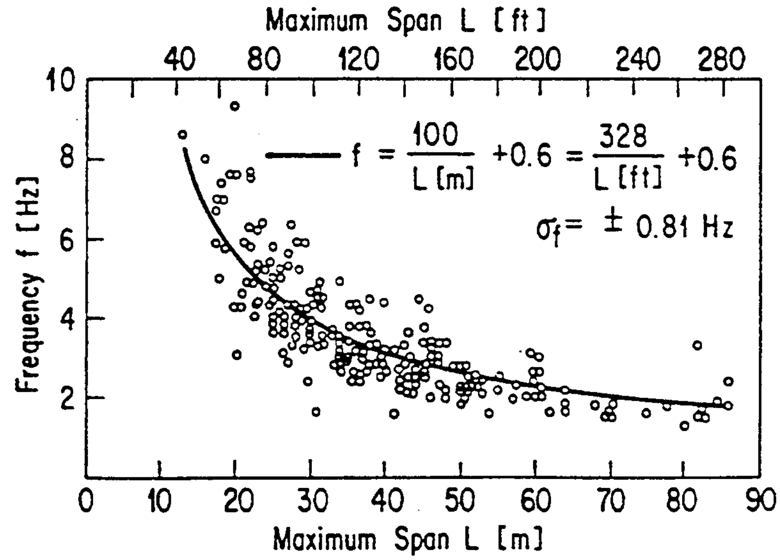

FIGURE 10 Fundamental frequency, f, as a function of the maximum span, L (from Cantieni 1984).

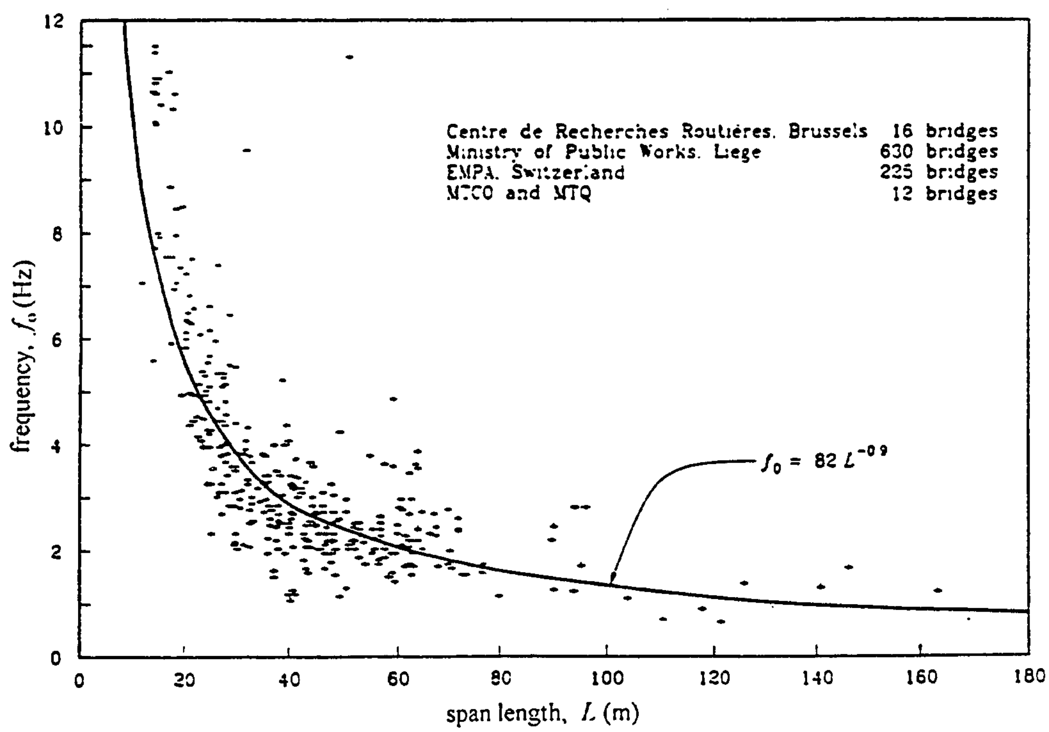

FIGURE 11 Fundamental frequencies versus span length (from Paultre et al. 1992).

was seen between measured frequencies and calculated frequencies obtained from analytical modeling of the bridges (Paultre et al. 1992). Reasonable correlation between the measured frequencies and bridge span was also seen, and several empirical equations describing this relation have been proposed (see Figures 10 and 11).

The dynamic response of any system is influenced by the system natural frequencies and the excitation frequencies. For highway bridges, somewhat periodic excitation may be induced by vehicle bounce due to roadway roughness. Thus, if the frequencies of the bridge and vehicle converge, a state of quasi-resonance can exist and the dynamic response induced may be large. For most heavy trucks, natural frequencies of the vehicle typically occur in two frequency ranges: (1) between approximately 2 and 5 Hz for the "body bounce" response and (2) between approximately 10 and 15 Hz for the "wheel hop" response (Cantieni 1983). The particular frequencies are truck and suspension dependent. The body bounce response can be excited by relatively long undulations in the roadway surface, and the wheel hop response by relatively short variations, for

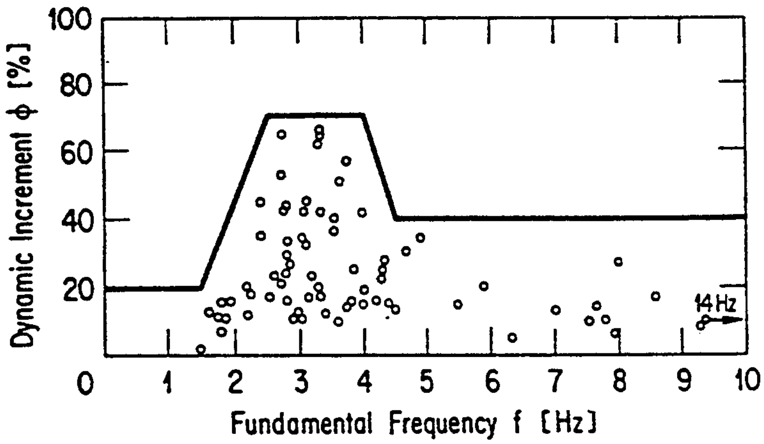

FIGURE 12 Dynamic increment as a function of fundamental frequency (from Cantieni 1984).

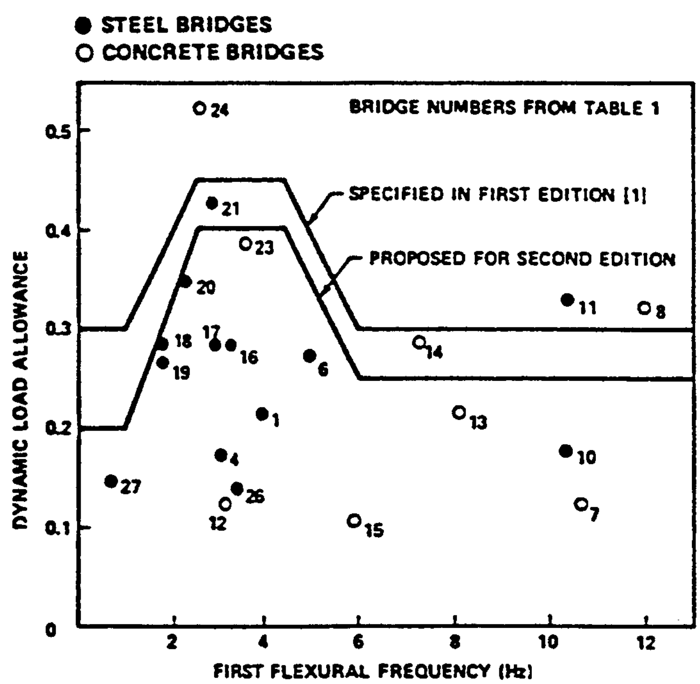

FIGURE 13 Dynamic load allowances from test and comparison with code (from Billing 1984).

example, a sharp irregularity in the roadway. However, depending on vehicle speed, irregularities may be effective in exciting both modes of response (Cantieni 1983). Numerous experimental studies have shown that largest dynamic response due to truck loading is obtained for bridges with natural frequencies in the range of 2 to 5 Hz, which corresponds to the body bounce response frequency range and therefore the potential loading frequency range of a truck (see Figures 12 and 13).

Bridge Span

The impact factor provisions in the AASHTO specifications have long been expressed as a function of bridge span. While some investigations have shown a general trend of decreasing impact in conjunction with increasing span (for example, see Figure 14 from Fleming and Romualdi 1961), other investigations have concluded that considerable scatter exists in the results and there is poor correlation of impact and span (see Figure 15 from Cantieni 1983). Some researchers have concluded that impact is not a function of bridge span (Coussy et al. 1989).

Bridge Damping

Damping values for bridges obtained from field testing vary considerably based on the method of testing, level of

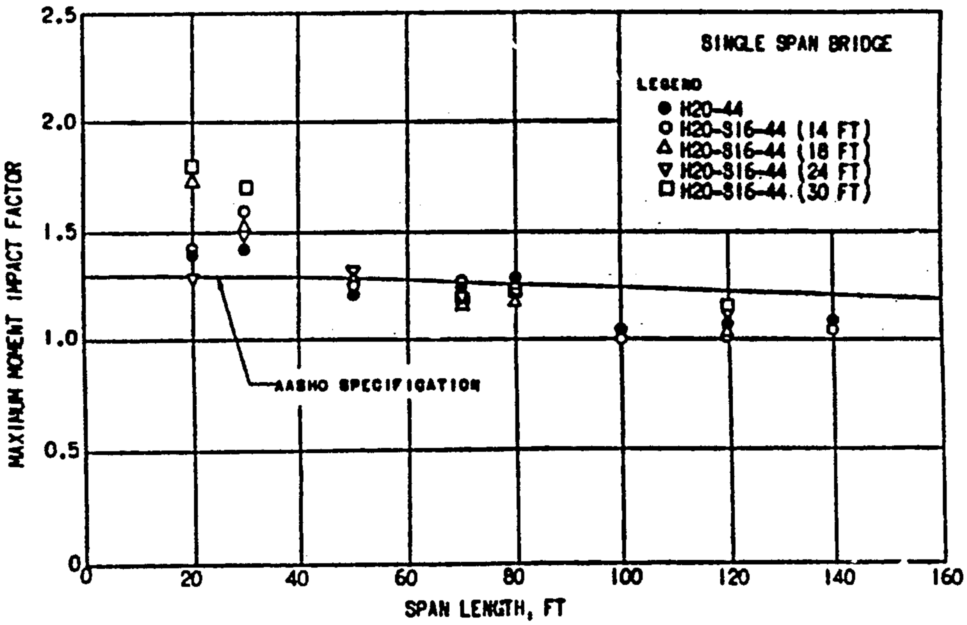

FIGURE 14 Comparison of computed impact with specifications (from Fleming and Romualdi 1961).

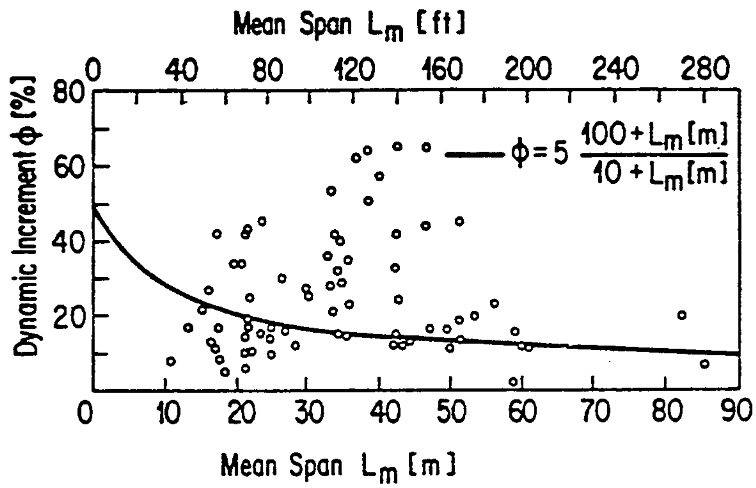

FIGURE 15 Dynamic increment as a function of the mean span (from Cantieni 1984).

loading, and different methods used for evaluating damping. Based on tests of bridges in Europe, Tilly (1978) reported damping values ranging from 2 to 10 percent for concrete bridges, 2 to 6 percent for steel bridges, and 5 to 10 percent for composite steel-concrete bridges. Billing (1984) reported damping values of 1 to 2.2 percent for prestressed concrete bridges and 0.4 to 1.3 percent for steel bridges tested in Ontario. The differences in the reported damping values from Tilly and from Billing for similar types of bridges may be due to different methods used when evaluating damping (Paultre et al. 1992). Ritter et al. (1995) reported damping values obtained from testing timber bridges of between 3 and 4 percent.

From basic dynamic principles, it would be expected that higher levels of damping would reduce the dynamic response in bridges. Similarly, low levels of damping in a bridge would be expected to result in high dynamic amplification. In analytical investigations of the effects of damping on dynamic response by Huang, Wang, and Shahawy (1992), it was found that damping affected impact differently at different locations within the bridge as a result of varying modal contributions.

Bridge Roadway Roughness and Approach Condition

Most studies of bridge dynamics have concluded that the roughness of the roadway surface of the bridge and its approaches have a significant influence on the magnitude of the dynamic response. In many experimental investigations, wooden

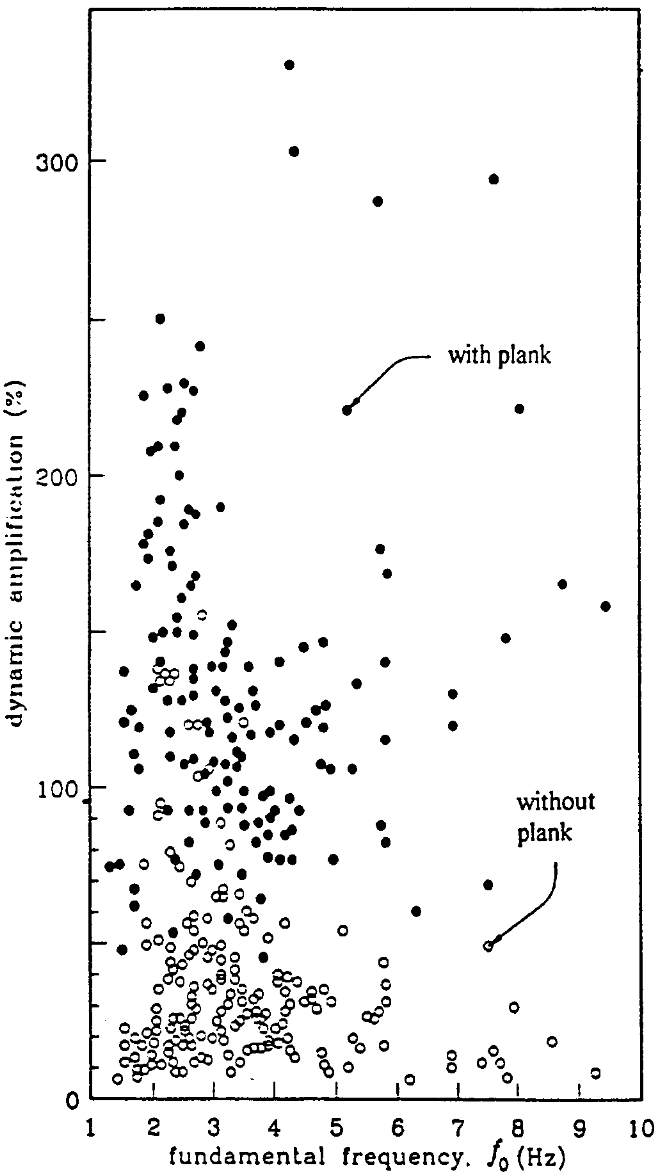

FIGURE 16 Effect of roadway roughness simulated by a plank on dynamic amplification (from Paultre et al. 1992).

planks were placed in the path of the test vehicle. As would be expected, dynamic response is higher with the planks (see Figure 16 from Paultre et al. (1992). It has also been observed that the planks can excite wheel hop in the test vehicles (see Figure 17 from Cantieni 1983), although excitation of the higher vibration mode associated with wheel hop is also speed dependent. Experimental tests have also shown that the most severe wheel impact forces are likely to occur adjacent to the bridge approaches, i.e., shortly after a vehicle enters the bridge (Tilly 1978).

The use of a plank to represent surface irregularities in the dynamic tests has been justified in that even a well-maintained bridge surface may have dropped objects or packed snow on the roadway. As an exception to this opinion, Paultre et al. (1992) postulated that, for multilane bridges, whose design loading is controlled by heavy vehicles simultaneously present in two or more lanes, the probability of having such a severe irregularity as a plank is so small as to be negligible.

Analytical studies of surface roughness have typically represented the road profile as a random process and described the surface roughness using a power spectral density function. Figure 18 (Wang, Shahawy, and Huang 1993a) shows the results obtained for an HS20-44 vehicle model with varying surface roughness representing ISO specifications for very good, good, average, and poor roadway surfaces. It can be seen from the figure that:

1. The impact forces increase for increasing roughness.
2. Vehicle speed affects the influence of roughness. For rougher surfaces, the faster the vehicle speed, the greater the impact forces. Conversely, vehicle speed has a much smaller effect on impact forces with better surfaces.

Bridge Type

A number of analytical investigations have been conducted into the dynamic response of particular types of bridges. Wang, Huang, Shahawy, and Huang (1996) investigated dynamic loading on simple-span multigirder highway bridges and concluded that, provided the roadway surface is in reasonably good condition, the total number of girders has little influence on the maximum impact factors for each girder. In a similar study, Huang, Wang and Shahawy (1992) analytically investigated impact in continuous multigirder bridges finding that impact at the interior supports was larger than at other locations due to the influence of higher natural frequencies. Yang, Liao and Lin (1995) found that the dynamic response of simple-span bridges was higher than that for similar continuous bridges.

Wang and Huang (1992), Huang and Wang (1992) and Khalifa (1993) conducted analytical investigations of the dynamic response of cable-stayed bridges. In their studies, it was found that, with a good road surface, calculated impact factors were generally less than 0.20. However, for rough surfaces, impact forces increased dramatically. The researchers also concluded that impact in a cable-stayed bridge is more complicated to assess than for beam/girder bridges and that it is likely that details associated with particular cable-stayed bridges will have a strong influence on dynamic response. Similar conclusions were reached by Chatterjee, Datta, and Surana (1994) when studying vehicular vibrations in a suspension bridge.

Wang, Huang, and Shahawy (1996) and Huang, Wang, and Shahawy (1995a) analytically investigated the dynamic response of continuous and cantilever thin-walled box girder bridges under multi-vehicle loading. It was found that the vibration characteristics of the continuous and cantilever box girder bridges are quite different. For cantilever bridges, the most important factor affecting impact is the vehicle speed, and cantilever bridges are much more susceptible to vibration than continuous bridges. This may be due to the abrupt change in loading due to span discontinuities, especially when no supported spans exist between cantilevers, as with bascule

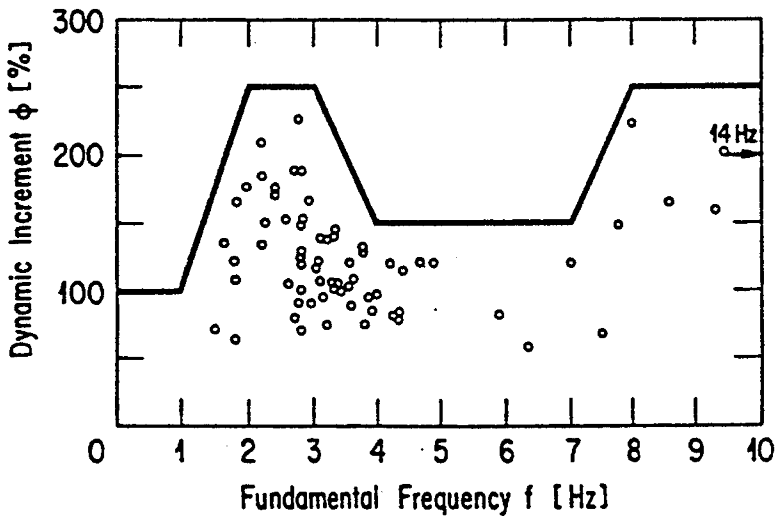

FIGURE 17 Dynamic increment for passages with a plank as a function of fundamental frequency (from Cantieni 1984).

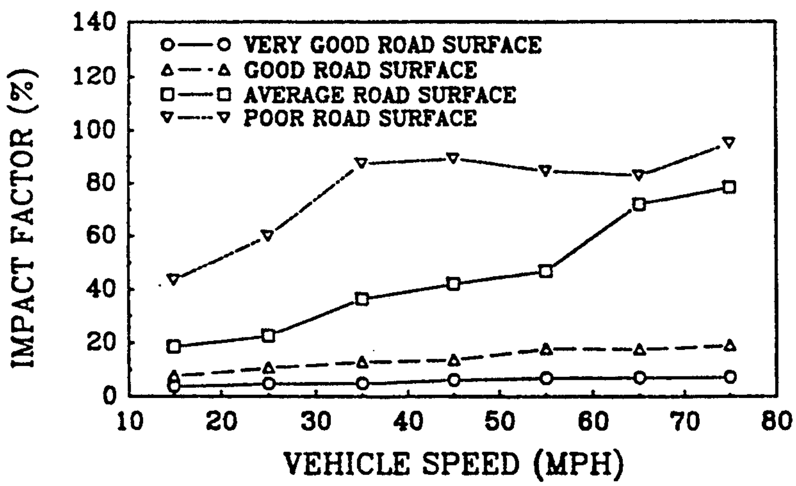

FIGURE 18 Impact results of tire forces (from Wang, Shahawy, and Huang 1993a).

bridges. For continuous bridges, both vehicle speed and surface roughness are significant. End diaphragms were found to provide lateral support and significantly reduce the response of the box girder bridges. The beneficial effect of a midspan diaphragm is relatively small.

Bridge Geometry

The effects on dynamic behavior of horizontally curved geometry has been studied analytically for box girder bridges by Galdos et al. (1993) and Schilling et al. (1992) and for I-girder bridges by Huang, Wang, and Shahawy (1995b). It was found that the dynamic response in horizontally curved bridges is influenced by centrifugal accelerations, thus, vehicle speed is particularly important. Impact forces are higher in the outer elements of the curved bridges. Impact forces are insensitive to curvature for radii greater than 4,000 ft (1219 m) and markedly influenced by curvature for radii less than 800 ft (244 m) (Galdos et al. 1993). In their study, the AASHTO provisions given in *Guide Specifications for Horizontally Curved Highway Bridges* were found to be conservative. However, if horizontal curvature was ignored and the provisions for straight girder elements, given in the *Standard Specifications for Highway Bridges*, were used in the design, then the calculated impact forces would be greater than predicted using the code (Galdos et al. 1993).

Vibration and impact in skewed steel bridges were studied in an analytical investigation by Wang, Huang, and Shahawy (1993b). The researchers concluded that impact forces in the girders increase with increasing angle of skew.

Bridge Material

Kawatani and Fukumoto (1992) found that the calculated impact forces of prestressed concrete and reinforced concrete highway girder bridges were almost the same as those for steel girder bridges. In contrast to the impact provisions in many code specifications, significant impact forces have been reported for timber bridges (Ritter et al. 1995 and Uppal and Rizkalla 1988). Despite different reported damping parameters, the construction material used in the bridge does not appear to have a significant influence on dynamic response.

Measured Bridge Response Parameter

The dynamic response of bridges has been measured by displacement transducers, strain gauges, and accelerometers. Displacement measurements are perhaps the easiest to obtain. When used, strain gauges are mounted to the sections of the bridge of interest, usually in peak bending moment regions. Accelerometers are used mainly when human perceptions of motion are of concern. It has been shown in numerous studies that dynamic load factors calculated based on strain measurements will consistently be smaller than factors calculated from displacement measurements (Paultre et al. 1992; Bahkt and Pinjarkar 1989; AASHO 1962).

Vehicle Parameters

Vehicle Speed

Vehicle speed, combined with surface conditions, influence the vibrational response induced in a vehicle traveling over a bridge. For heavy commercial vehicles, the vibrational modes of interest are body bounce at frequencies between 2 to 5 Hz and wheel hop at frequencies of between 10 and 15 Hz. The vehicular vibration modes excited will influence the dynamic response of the bridge, as discussed earlier. Figure 18 illustrates the effects of vehicle speed and surface condition on impact factor resulting from an analytical investigation. For smooth roadway surfaces, the impact factor increases only slightly with vehicle speed, while for increasingly rough surfaces the impact factor increases rapidly with increasing vehicle speed.

Vehicle Weight

Many studies have shown that, as the weight of the crossing vehicle increases, the magnitude of the dynamic response expressed as a percentage of the static load decreases. Experimental results illustrating this effect are shown in Figure 19 (Billing and Green 1984) and Figure 20 (Chan and O'Conner 1990). Analytical investigations of the effects of vehicle weight on dynamic response have reached similar conclusions, as shown in Figure 21 (Wang, Huang, and Shahawy 1993a) and Figure 22 (Nassif and Nowak 1995). Figure 21 also shows that the shorter the span, the more rapidly the impact factor will increase with lessening weight.

The explanation of why the impact factor decreases with heavier vehicles is that, while dynamic forces increase with increasing vehicle weight, the static load increases more rapidly with increasing weight. Thus, the impact ratio of dynamic force to live load decreases with increasing vehicle weight. An important conclusion from this observation is that impact factors obtained from the measured dynamic response of lightly loaded vehicles will be relatively large. However, these factors are irrelevant from a design perspective, as they correspond to low stress levels in the bridge (Bahkt and Pinjarkar 1989).

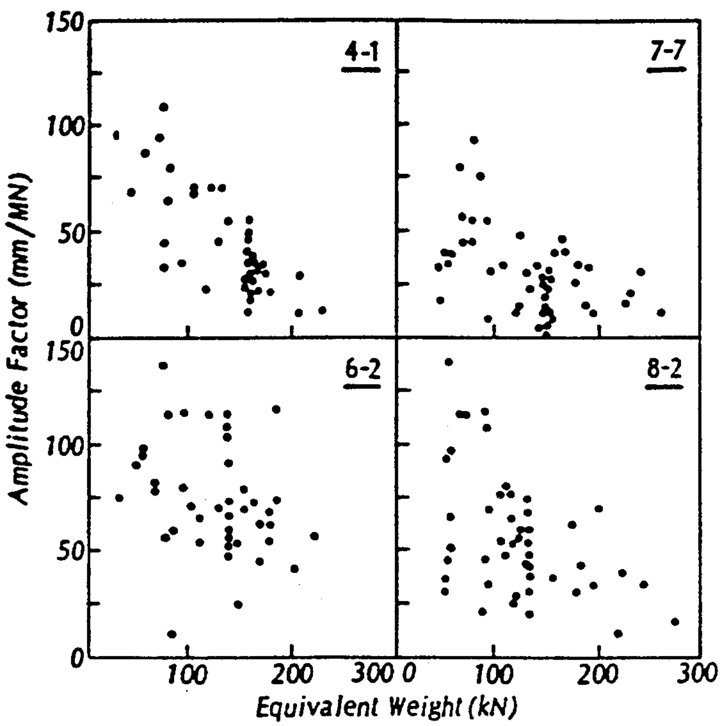

FIGURE 19 Amplitude factor versus equivalent weight (from Billing and Green 1984).

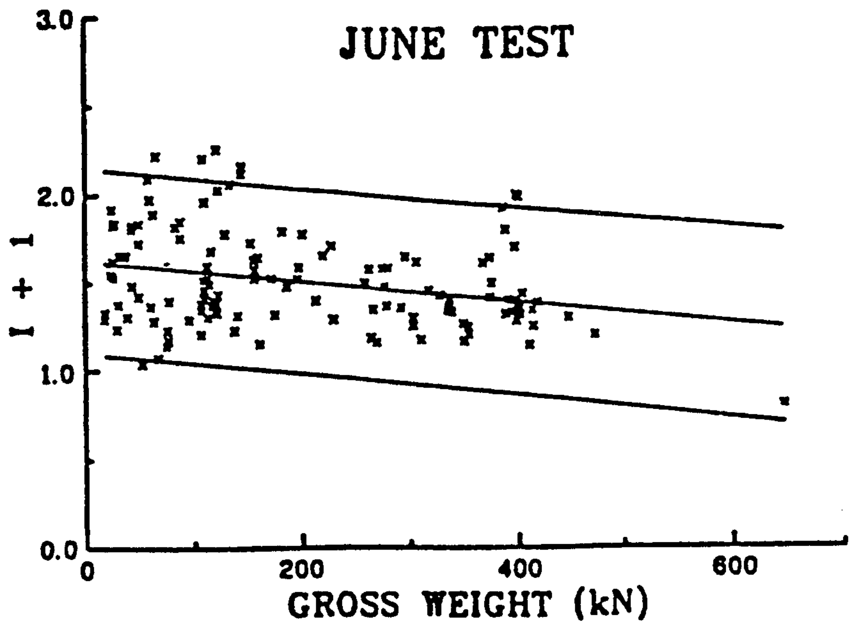

FIGURE 20 Impact + 1 versus vehicle weight (from Chan and O'Connor 1990).

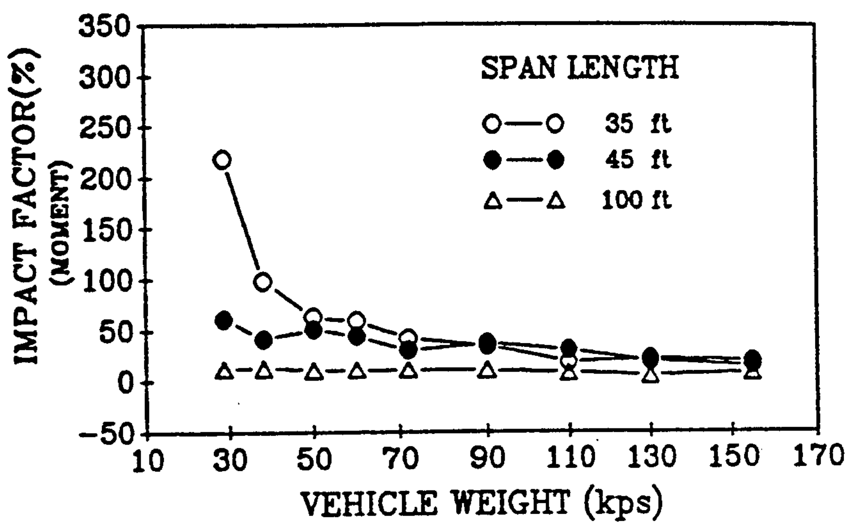

FIGURE 21 Effects of vehicle weight on impact (from Wang, Huang, and Shahawy 1993a).

As previously discussed, the dynamic load factor decreases with increasing vehicle weight. However, studies have also shown that the dynamic load factor is a function of the number and spacing of axles on the vehicle, which is related to vehicle weight. The dynamic response from the individual axle loads will combine or interfere with each other, depending on the arrangement of the axles. Hwang and Nowak (1991) analytically determined the dynamic load factors for various truck types and reported that the largest factors were for a single truck and lowest for a tractor-trailer. Nowak, Hong, and Hwang (1990) concluded that, in general, the dynamic load is lower for a larger number of axles. However, Nassif and Nowak (1995) later reported that, excluding two-axle vehicles, four- and five-axle trucks cause the largest dynamic load factors. Wang, Shahawy, and Huang (1993a) found analytically that, for a HS20-44 tractor-trailer vehicle, the impact factors resulting from the tractor axle are much higher than those from the trailer axle. They also found that impact factors were highest when the spacing between the tractor and trailer axles was 4.5 m (15 ft) and 8.2 m (27 ft).

Number of Vehicles

Studies have shown that the dynamic load factors associated with multiple vehicles are lower than those for single vehicles, as shown in Figure 23 (Nowak, Hong, and Hwang 1990). This is most likely because the total static load is larger (similar to having a heavier vehicle) compared to the associated dynamic load, and the dynamic responses from the two

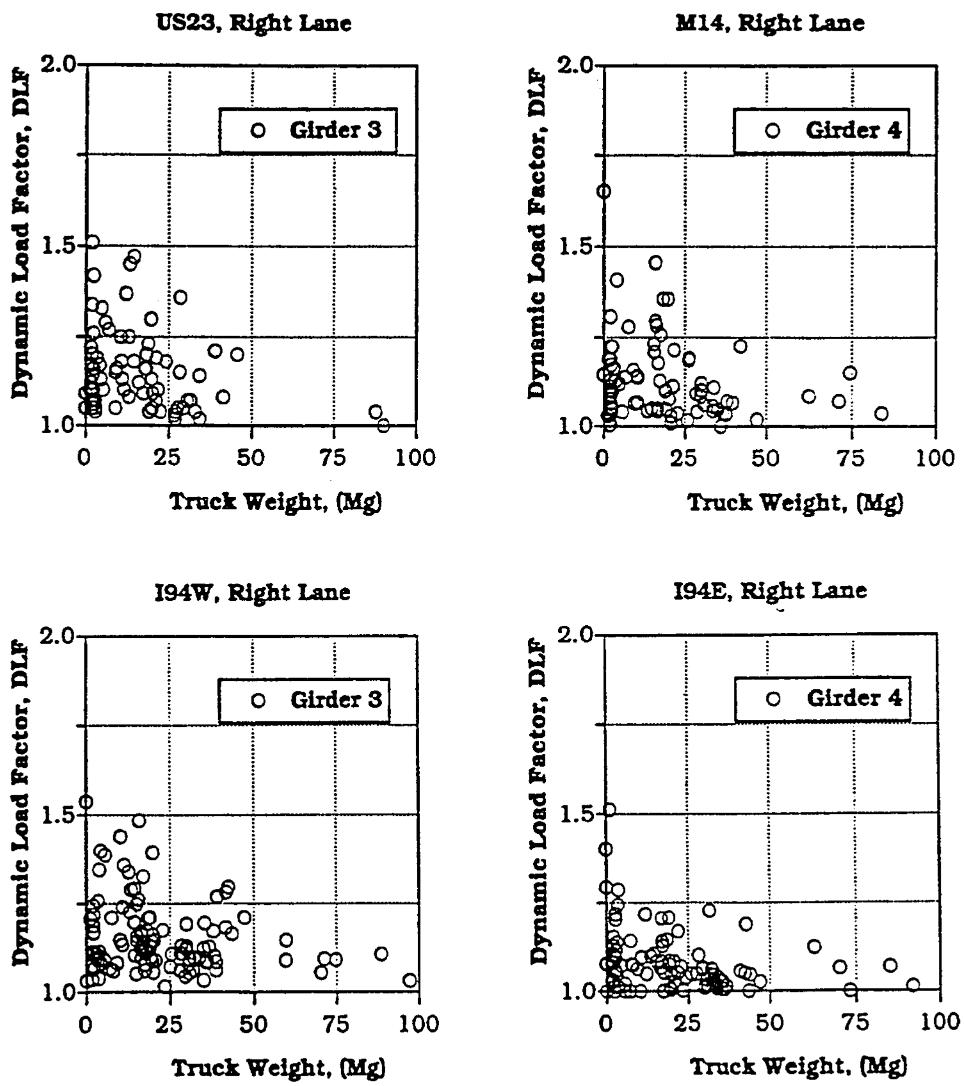

FIGURE 22 DLF versus truck weight (from Nassif and Nowak 1995).

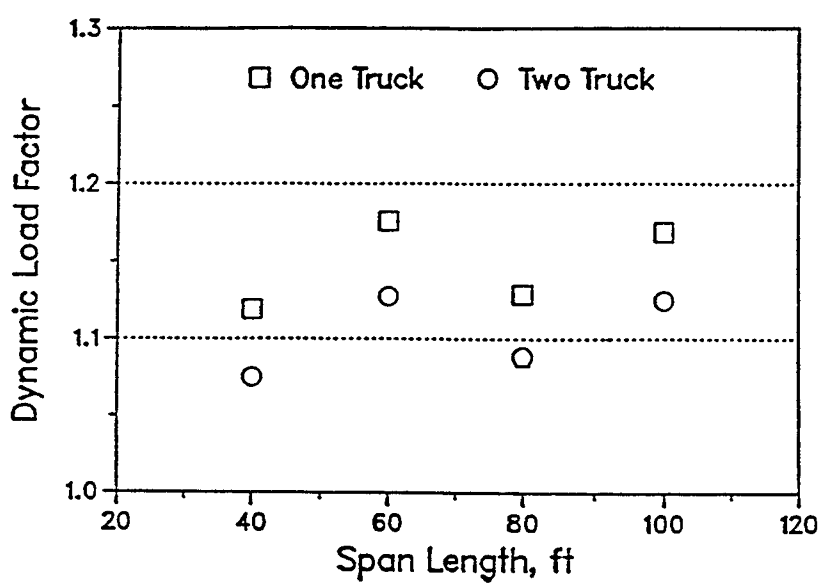

FIGURE 23 Dynamic load factor for single- and two-axle trucks (from Nowak, Hong, and Hwang 1990).

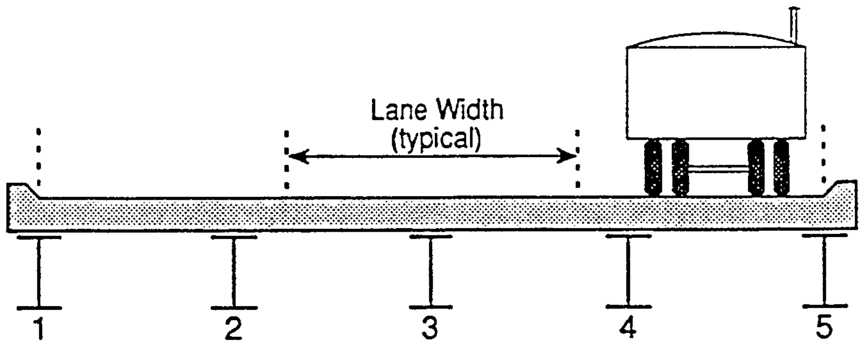

FIGURE 24 Cross-section of a three-lane bridge (from Bahkt and Pinjarkar 1989).

individual vehicles are likely to be at least somewhat out of phase with each other (Bakht and Pinjarkar 1989).

Vehicle Position

The position of a vehicle on a bridge, whether symmetrically or unsymmetrically located, influences the modes of vibration that will be excited by passage of the vehicle over the bridge. Further, the location(s) where dynamic response is monitored relative to the position of the vehicle is very important in drawing conclusions on dynamic load allowances. As an illustration of the issues, Bakht and Pinjarkar (1989) considered a three-lane bridge as shown in Figure 24. Assume that all five girders are instrumented for dynamic response measurement and that the vehicle is positioned in the right-hand lane. Girders 1 and 2 will carry a relatively small portion of the static load. However, the dynamic amplification of the small static load carried by these girders is likely to be relatively large. Correspondingly, the dynamic load factor based on the measured response in these girders will be large. However, this factor is not relevant as far as bridge design loading is concerned. Numerous studies have observed that dynamic factors are larger in exterior girders and at points far away from the load as a result of this phenomenon.

Vehicle Suspension

As discussed earlier, heavy vehicles tend to respond in two primary vibrational modes: body bounce in the 2 to 5 Hz range and wheel hop in the 10 to 15 Hz range. In turn, these primary frequency response ranges have been shown to interact with bridges having fundamental frequencies in the same ranges. However, the vehicle frequency ranges are a function of the suspension systems common in these type of vehicles. Heywood (1995) observed that new generations of highly damped air suspension systems are gaining in popularity. Through field studies, he found the body bounce frequencies in vehicles with air suspensions to be lower than those for steel leaf-spring suspensions, with measured frequencies in the 1.5 to 2 Hz range. In general, dynamic response was less for the air-suspended vehicles than for steel-suspended vehicles, unless vehicle frequencies coupled with the bridge frequencies. Heywood also noted that worn dampers in the suspension systems dramatically increased the dynamic wheel forces.

CHAPTER FOUR

DESIGN PROVISIONS FOR DYNAMIC LOAD EFFECTS

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

Bridge Design

1996 Specifications

The 1996 AASHTO *Standard Specifications for Highway Bridges* (AASHTO 1996) addresses the dynamic effects generated by vehicular traffic loads by the mandatory consideration of "impact" as outlined in Section 3.8 and by recommended limitations on flexural member depth and on deflections as outlined in Section 8.9 for reinforced concrete, 9.11 for prestressed concrete, and in 10.5 and 10.6 for structural steel.

Impact Provisions—The impact provisions increase the effects of the live load, both lane and truck loads, and apply to all those structural elements of a bridge that are above the ground. Included are the superstructure and those parts of the piers that are above ground. Excluded are the abutments, retaining walls, footings, piles (except those parts of pile bents that are above ground), timber structures, and structures having three or more feet of cover.

The amount of increase in live load effect (e.g., stress) is defined by the formula:

$$I = 50/(L + 125) \tag{8}$$

where I is the impact fraction, which is not to exceed 30 percent, and L is the length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

Background for the Impact Provisions—The impact factor, in its current form, is based on the recommendations of a joint AREA and AASHO committee in 1927 (ASCE 1931). This period saw the development of several versions of impact formulae that decreased with increasing span. In 1931, the final recommendations of a special ASCE committee on highway bridge impact (ASCE) were to use:

$$I = 50/(L + 160) \tag{9}$$

with a cap at 25 percent. The various expressions considered by the committee are shown in Figure 25. In the figure, the final recommended equation is slightly less conservative than that recommended in 1927. It is also pointed out in the discussion following the 1931 report that the authors of the formula for Curve 3 of Figure 25, intended for the formula to apply all the way to zero length span (i.e., the maximum impact would be 40 percent).

The basis of the impact formulae is rooted in railway engineering where the dynamic amplification of forces is related to the periodic forces due to unbalanced driver forces (hammer blow) of steam locomotives. This type of loading is no longer

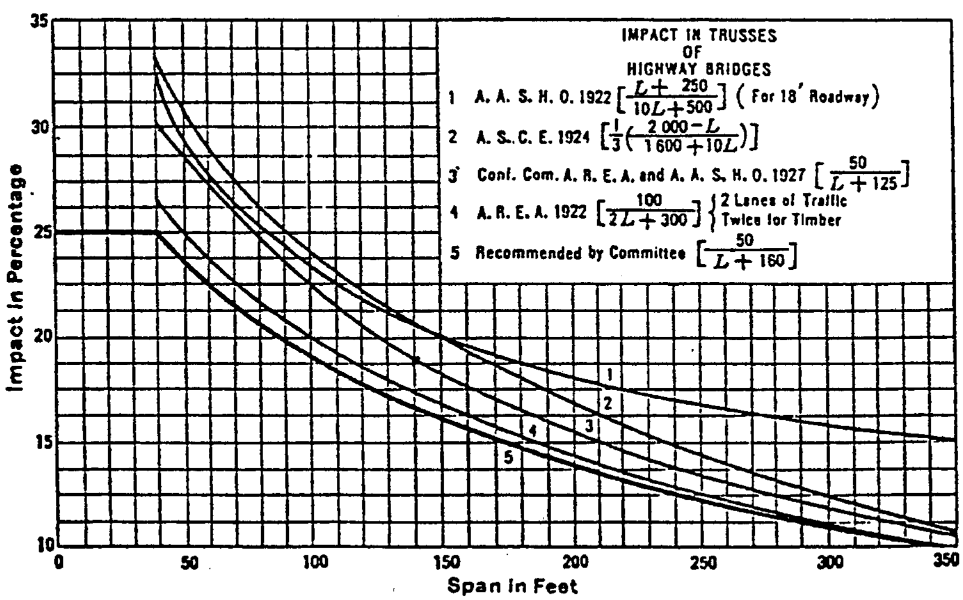

FIGURE 25 Early impact formulae (from ASCE 1931).

as significant as it was in the 1920s, but it is still considered in the design of railway bridges in the United States (AREA 1996).

Even though the magnitude of the impact factors for railway loading was considered inappropriate for highway bridges, the decrease with span length was considered by the committee (ASCE 1931), albeit with some reservation. In fact, the report states, "Existing data are too meager to establish a relation between impact and span length." The apparent decrease in impact results from the observation that the dynamic load increment appears somewhat independent of span, while the static effect of loading increases with span. Thus the impact factor, which is typically defined as the ratio of the maximum dynamic plus static effect to the static effect, decreases as the span increases.

The committee further concluded that for shorter spans [less than 12 m (40 ft)], including bridge floors and floorbeam suspenders, a maximum impact factor of 25 percent was appropriate. Thus a cap of 25 percent was recommended for the design expression. However, it was recognized that the presence of obstructions on the bridge could cause the impact factor to markedly increase. The recommended values were acceptable only if the roadway surface was fairly smooth. It should be noted that the surface roughness effects considered by the committee were primarily based on solid-tire trucks crossing 25- and 50-mm (1- and 2-in.) tall obstructions at 15 mph. Some pneumatic-tire trucks were included and the committee indicated that impact fractions were generally under 30 percent.

Over the years, researchers have continued to study the impact phenomenon, and improved formulae have been proposed to the AASHTO Committee, but no modifications have been adopted into the *Standard Specifications*. However, other AASHTO specifications and guide specifications have used different methods for handling impact. Several of these (AASHTO 1994; AASHTO 1993; AASHTO 1989) are discussed later in this chapter.

Member Depth and Deflection Limit Provisions—In addition to the direct inclusion of impact effects in Section 3.8, the *Standard Specifications* suggest limits on the depths of flexural members and suggest limits for their deflections.

The limits on member depth are given in Table 8.9.2 of the *Standard Specifications* for reinforced concrete, there are no direct limits on member depth for prestressed concrete, and for structural steel elements, including trusses and composite members, limits are provided in Section 10.5.

The deflection limits are likewise given in the material chapters, however, the limits are the same for all three materials. The specifications suggest a maximum deflection due to service live load plus impact of 1/800 of the span, except where significant pedestrian traffic also uses the bridge and the limit is 1/1000 of the span. These limits apply to both simple and continuous spans. For cantilevers, the ratios are 1/300 and 1/375 of the span.

Background for the Member Depth and Deflection Limit Provisions—The deflection provisions have been included for years in the *Standard Specifications* and apparently were incorporated early in the century to both limit deflection and to control service load vibrations (ASCE 1958). The recommended limits for deflection control for reinforced concrete and prestressed concrete members has come about in the last 20 years.

In 1958, the ASCE Committee on Deflection Limitations of Bridges (ASCE 1958) reviewed the deflection limits of the 1953 AASHO *Standard Specifications*. The committee recommended that the *Specifications* remain unchanged, even though the original basis of the provisions was not determined and that the limits were somewhat arbitrary and rooted in railway engineering. The committee did recognize the need to define what physical parameters related to objectionable vibration and recommended that studies be undertaken to develop appropriate criteria for vibration control. Since 1958, the body of literature on this topic has expanded greatly, and information relevant to transportation structures can be found in OMTC (1991), OMTC (1983), OMTC (1979) and AISI (1974).

1994 LRFD Specifications

The AASHTO *LRFD Bridge Design Specifications* (AASHTO 1994) is a departure from the *Standard Specifications*. The *LRFD* provisions for loading were developed using calibration with existing data. The loading provisions are given in Section 3 and the impact provisions are contained in Section 3.6.2. Additionally, criteria for control of deflections are contained in Section 2.5.2.6, but these criteria are explicitly defined as optional.

The *LRFD Specifications* are not simply a conversion of the *Standard Specifications'* "Strength Design Provisions." Instead, it has been developed based on defined limit states, including service, strength, and extreme events, and the code has been calibrated to provide, in as much as practical, consistent reliability of bridge structures (Nowak 1995). To this end, the load combinations, load factors and the loads and "impact factors" have been changed.

Dynamic Load Allowance Provisions—Section 3.6.2 provides the provisions for "impact." One of the departures from previous *Standard Specifications* is the use of the terminology: "dynamic load allowance" in place of impact. It has been recognized for years that the term "impact factor" or "impact fraction" is too restrictive and not reflective of the purpose of accounting for all vehicle-induced dynamic effects (ASCE 1981; OMTC 1979).

The new provisions retain the format of accounting for vehicle-induced dynamic effects by simply scaling the static load effects. However, only the design truck or tandem (axles) static loading effects, exclusive of centrifugal and braking effects, are increased; neither the uniform lane load nor pedestrian loads are increased. The increase is also simply a single percentage for a given component, and is thus independent of span. The dynamic load allowances, IM, are 75 percent for deck joints for all limit states, 15 percent for other components (i.e., elements other than deck joints) for fatigue and fracture limit states, 33 percent for other components for all limit states, except fatigue and fracture. The dynamic load allowance along with the live load are subject to reduction due to the presence of multiple vehicles. The multiple presence factors are outlined in Section 3.6.1.1.2.

The dynamic increases are not applied to foundation elements (piles, footing, etc.) that are completely buried, and the increases are also not applied to retaining walls, provided such walls do not support vertical reactions from the superstructure. The exclusion is allowed since the presence of soil provides damping that reduces the amplitude of dynamically induced forces. Additionally, dynamic load allowance, *IM*, for buried structures and culverts is given as:

$$IM = 40\,(1.0 - 0.125\,D_E) \qquad (10)$$

where D_E is the minimum depth of cover of the structure in feet.

For wooden components and wooden bridges, the dynamic allowance may be reduced by 50 percent due to the higher damping and improved short-term loading properties known to exist in wood.

Another addition in the *LRFD Specifications* relative to the previous *Standard Specifications* is the permission to use dynamic analysis to determine or refine the dynamic load allowance. These provisions are given in Section 4.7.2.1 and allow the designer to reduce the dynamic load allowance by up to 50 percent if substantiated by proper analysis. For such an analysis the designer and the owner must consider and agree on several parameters. For instance, the surface roughness shall be considered, as must vehicle speed and vehicle dynamic characteristics (e.g., suspension, sprung mass, etc.). The limit of the reduction to 50 percent is meant to account for the fact that actual wearing surface conditions are difficult to accurately anticipate over the life of the structure.

Background for the Dynamic Load Allowance Provisions—Since the *LRFD Specifications* development reopened a number of issues regarding the connection between code values and observed and calculated data, the new dynamic load allowance provisions were largely based on recent field data and numerical simulation results that attempted to account for the many variables that are germane to dynamic response.

The basis of the new dynamic load allowance provisions is outlined in the notes for National Highway Institute Course 13061 (NHI 1995) and by Nowak et al. (1990), Nowak (1995), and Hwang and Nowak (1991), and the discussion here is based on these references. In the new *LRFD Specifications*, not only the dynamic load allowance (previously impact) provisions changed, the live load model changed significantly as well.

During comparison studies of overload or exclusion vehicle effects with that of the AASHTO HS20 and HS25 vehicles, it was discovered that the AASHTO vehicles, as they have been historically applied to bridges for design, did not give close results. Several alternate proposed loads were considered in these comparisons and two of these proposed loads gave much better fits with the exclusion vehicle results, and so these were adopted for the new *LRFD Specifications*. The loads are (1) a pair of 10.9-Mg (24-kip) tandem axles acting simultaneously with the basic AASHTO 2.85-kN (0.640-kip/ft) lane load or (2) the HS20 vehicle acting simultaneously with the lane load. The new impact provisions thus had to provide a rational and realistic treatment of these new load types.

Because field testing often does not facilitate the exhaustive investigation of all the important variables involved, particularly at design load levels, a series of numerical simulations were run to study the effects of principal variables (Hwang and Nowak 1991; Nowak et al. 1990). The analyses included surface roughness, two-dimensional models of the HS20 truck including the suspension, weight of the truck, span length, steel and concrete girder simple-span bridges, axle spacing, and number of trucks present. The results were given as dynamic load factor (DLF), which was defined as the maximum dynamic deflection divided by the maximum static deflection at midspan.

The results of this work gave the following conclusions:

1. The dynamic amplification drops as the vehicle weight increases, as shown in Figure 26.
2. The speed of the vehicle is not a primary variable, although some correlation with speed exists, as seen in Figure 27.
3. Roughness increases dynamic amplification, but more so for lighter vehicles, as seen in Figure 28.
4. More than one vehicle decreases the amplification, as seen in Figure 29.

Based on these results, and considering the large body of earlier data, it was decided that the basic dynamic load allowance would be 33 percent. This would be a constant value applied to the truck load only and no allowance would be applied to the lane load. In recognition of the fact that the lane load also produced some dynamic effects, albeit much smaller than those from a single truck, it was argued that the true amplification of a single truck, which is approximately 25 percent, would be increased to the 33 percent.

The impact factor specified for joints, *IM* of 75 percent, is based on test results from Europe and has recently been verified in the United States during testing of modular bridge joints (Dexter et al. 1997). While the high impact factor from the U.S. tests is partially due to the dynamic characteristics of modular joints themselves, it is probably not unduly conservative to apply the 75 percent factor to all types of joints.

Deflection Limit Provisions—Section 2.5.2.6 provides a general overview of deformation control in bridges. The Commentary alludes to the use of deformation limits to limit undesirable vibrations from dynamic loading, although the Commentary also recognizes that today there are better, perhaps more direct, methods for controlling vibrations, and it even references the reader to the 1983 Ontario Highway Bridge Design Code (OMTC 1983) for criteria.

Section 2.5.2.6.2 contains the *LRFD* Criteria for Deflection and is considered optional, except for orthotropic decks for which the criteria are mandatory. The section provides both deflection limits for live loads and span-to-depth ratio limits. The deflection limits are the same as those that have been included in the *Standard Specifications* for years, and the

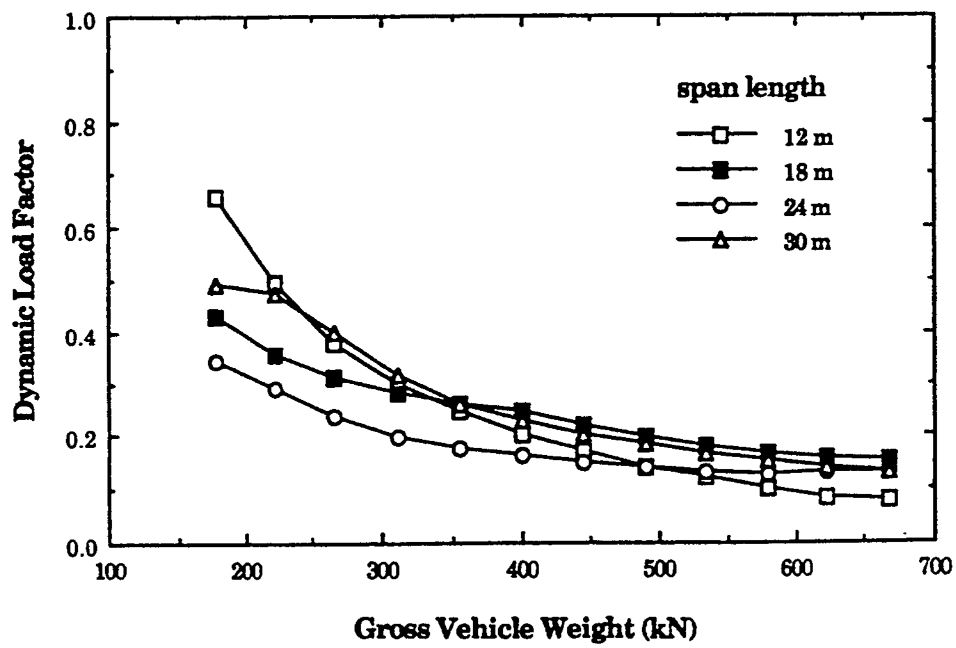

FIGURE 26 Dynamic load factor versus weight for steel girder bridges (from Hwang and Nowak 1991).

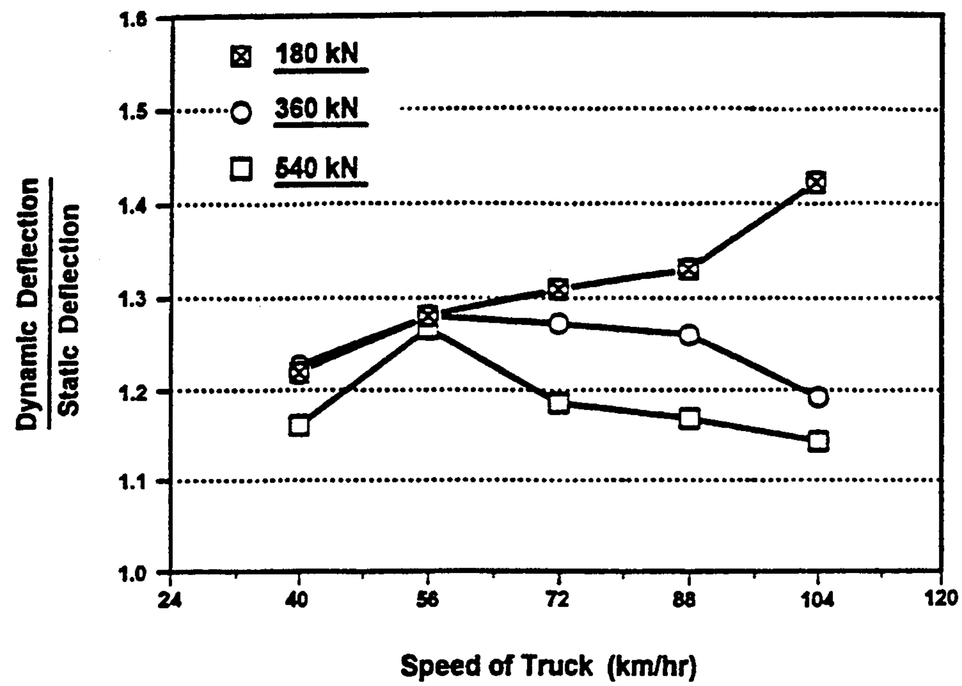

FIGURE 27 Effect of speed on dynamic load effect for steel girder bridges (from NHI 1995).

span-to-depth ratios are essentially the same ones that have been included in the *Standard Specifications* as well. The section also requires that the dynamic load allowance be included in the live load for which deflections are being checked.

Span-to-depth ratios have historically been allowed as an alternate to checking deflections for the control of vibrations and excessive deformations. For the most part, the criteria provided for span-to-depth limits in Table 2.5.2.6.3.1 of the *LRFD Specifications* are identical to previous values given in the *Standard Specifications*. There are several new additions to the span-to-depth limits, including limits for prestressed concrete and modified limits for continuous steel beams.

The Commentary explains that the provisions "permit, but do not encourage, the use of past practice for deflection control." The provisions are included in the *LRFD Specifications* due to continued requests for some guidance in the area.

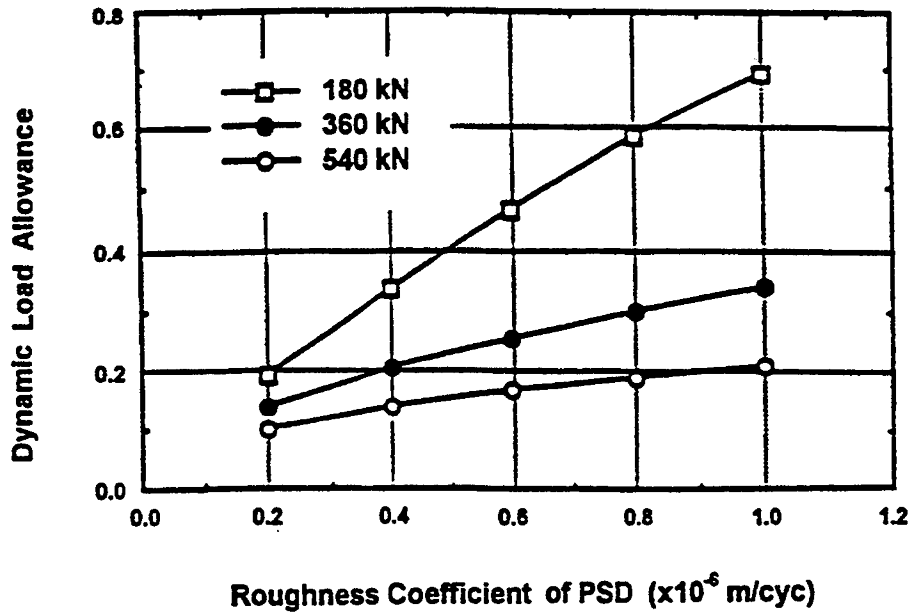

FIGURE 28 Effect of surface roughness on DLA (from NHI 1995).

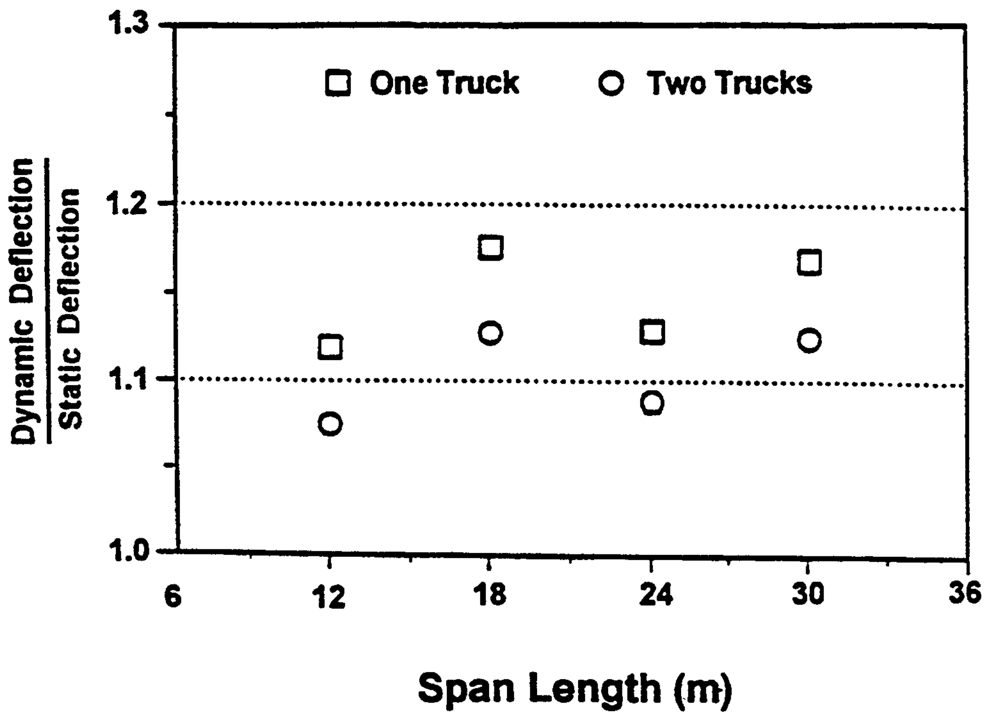

FIGURE 29 Effect of number of vehicles on dynamic response (from NHI 1995).

Load Rating Existing Bridges

The 1989 *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges* (AASHTO 1989) provides proposed guidelines for load rating existing bridges. In the load rating *Guide Specification*, dynamic effects of loads are included as they are in the *Standard Specifications*, as an increase in live load effect, in other words, a scaling of the static value. Impact is given solely as a function of the surface roughness condition of the bridge wearing surface: "For smooth approach and deck conditions, the impact may be taken as 0.10. For a rough surface with bumps, a value of 0.20 should be used. Under extreme adverse conditions of high speed, spans less than 12 m (40 ft) and highly distressed pavement and approach conditions, a value of 0.30 should be taken." Alternately, the *Guide Specification* ties impact to the condition of the wearing surface as shown in Table 3 excerpted from the *Guide Specification*. Both methods are subjective and require judgment on the part of the evaluator.

TABLE 3

IMPACT AS A FUNCTION OF WEARING SURFACE FOR LOAD RATING (from AASHTO 1989)

	Condition of Wearing Surface		Impact
1	Good condition	No repair required	0.1
2	Fair condition	Minor deficiency, item still functioning as designed	0.1
3	Poor condition	Major deficiency, item in need of repair to continue functioning as designed	0.2
4	Critical condition	Item no longer functioning as designed	0.3

The impact values listed are less than those given by the *Standard Specifications* due to conservative conditions that the *Standard Specifications* are based on, according to the *Guide Specification*. Additionally, it is recognized that under "enforced speed restriction" the impacts may be reduced, although no guidance is given.

The surface roughness condition is one that may change over the life of the structure and it is therefore difficult to assign one value for the structure. However, after a bridge is constructed, conditions that were not known to the designer may be apparent to the inspector/evaluator and thus a roughness-dependent impact factor may be appropriate. For instance, the Commentary to the *Guide Specification* suggests that the approach conditions be considered in the roughness assessment. A condition could exist that chronically sets vehicles into vertical oscillation before entering the bridge. This should be accounted for in the assessment.

Finally, it should be recognized that the language of the impact provisions in the *Guide Specification* allow, but do not require, the evaluator to drop the impact factor below that given in the *Standard Specifications*. Alternately, for longer span structures, the *Guide Specification* may suggest a larger impact factor than used in the original design.

Horizontally Curved Bridges

The 1993 AASHTO *Guide Specification for Horizontally Curved Highway Bridges* (AASHTO 1993), which essentially applies only to curved steel bridges, has its own set of impact or dynamic load allowance provisions for several types of curved bridge. The *Guide Specification* addresses both allowable stress design and load factor design methodologies in Parts I and II, respectively. Within each part, individual specifications are given for I-girder and for box girder bridges, although the provisions for impact are the same for both parts of the *Guide Specification*.

For I-girder bridges, the *Guide Specification* requires the use of the same impact adjustments to loading as used in Section 3.8.2 of the *Standard Specifications*. However, the Commentary for I-girder bridges has a detailed discussion of a proposed specification, and these are discussed in this section. For box girder bridges, the *Guide Specification* requires the use of a different set of impact provisions from those of the *Standard Specifications*.

I-Girder Bridge Proposed Specification—Commentary Section 1.3 (B) of Part I describes the proposed provisions. The form of the proposed provisions is similar to those traditionally used in that the dynamic live load effect is calculated by scaling the static load effects upward. However, centrifugal forces are included in the proposed provisions, whereas they have not been traditionally. The proposed impact factors and the limits for which they apply are given in Table 4. In the table, L is the span length; R_C is the radius of the centerline of the bridge; and v is the vehicle speed. If the given limits on parameters are exceeded, then a dynamic analysis is suggested. It is seen that different impact factors are applied to different stress resultants and to deflections. This is a departure from present practice in the *Standard Specifications*.

TABLE 4

PROPOSED IMPACT FACTORS FOR CURVED BRIDGES (from AASHTO 1993)

Quantity	Impact Factor, I
Reactions and shear forces	0.30
Moments in longitudinal girders	0.25
Torsional moments in longitudinal girders	0.40
Moments in slab	0.20
Bimoments in longitudinal girders	0.25
Forces and moments in diaphragms	0.25
Deflections	0.25

50 ft $\leq L \leq$ 200 ft. 200 ft $\leq R_c \leq$ 1,000 ft. $v \leq$ 70 mph. Number of I-girders $\leq$ 6. Number of continuous spans $\leq$ 2. (Weight of Vehicle)/(Weight of Bridge) $\leq$ 0.6.

Box Girder Bridge Provisions—Section 1.25 (B) of Part I describes the provisions for impact for box girder bridges. Unlike the proposed provisions for I-girder bridges, the impact provisions for box girder bridges are mandatory. The format is similar to the I-girder provisions in that explicit limits of applicability are given, although these limits are dependent on the stress resultant that is being considered. Also, the impact factor is typically a piece-wise function of span length, as can be seen in Table 5. Again, if the limits are exceeded then a dynamic analysis to determine dynamic load effects is recommended.

ONTARIO MINISTRY OF TRANSPORTATION

The Ontario Highway Bridge Design Code (OHBDC) (OMT 1991) has seen significant evolution in the past 18 years with respect to its dynamic load allowance (DLA). Based on a substantial evaluation program begun in the late 1960s and other work dating back into the 1950s (Csagoly

TABLE 5

IMPACT FACTORS FOR CURVED BOX GIRDER BRIDGES (from AASHTO 1993)

Quantity	Impact Function	Limits L (ft)	Limits R_c (ft)
Primary Bending Moment	$I_M = 0.320$	$L \leq 80'$	
	$I_M = 0.360 - L/2{,}000$	$80' < L \leq 200'$	$160' \leq R_c \leq 800'$
	$I_M = 0.260$	$200' < L$	
	$I_M = 0.285$	$L \leq 80'$	
	$I_M = 0.315 - L/2{,}667$	$80' < L \leq 200'$	$800' < R_c$
	$I_M = 0.240$	$200' < L$	
Torsion	$I_T = 0.275$	$L \leq 80'$	$160' \leq R_c \leq 800'$
	$I_T = 0.291 - L/5{,}000$	$80' < L$	
	$I_T = 0.260$	$L \leq 80'$	$800' < R_c$
	$I_T = 0.308 - L/1{,}667$	$80' < L$	
Shear	$I_S = 0.215$	$L \leq 80'$	
	$I_S = 0.243 - L/2{,}857$	$80' < L \leq 200'$	$160' \leq R_c$
	$I_S = 0.173$	$200' < L$	
Reactions	$I_R = 0.275$	$L \leq 80'$	
	$I_R = 0.310 - L/2{,}222$	$80' < L \leq 200'$	$160' \leq R_c \leq 800'$
	$I_R = 0{,}220$	$200' < L$	
	$I_R = 0.225$	$L \leq 80'$	
	$I_R = 0.253 - L/2{,}857$	$80' < L \leq 200'$	$800' < R_c$
	$I_R = 0{,}183$	$200' < L$	
Deflections	$I_D = 0.255$	All L	$160' \leq R_c \leq 800'$
	$I_D = 0.255$	$L \leq 80'$	
	$I_D = 0.271 - L/5{,}000$	$80' < L \leq 200'$	$800' < R_c$
	$I_D = 0.231$	$200' < L$	

and Dorton 1978), the Ontario Ministry of Transportation and Communication (OMTC) broke ranks with AASHTO and developed a new methodology for handling impact or dynamic load allowance (DLA) as the traditional bridge vehicle impact factor came to be known. The evolution of these provisions for DLA are reviewed in this section.

The OHBDC was first issued in 1979 (OMTC 1979) as the result of the Ontario Ministry of Transportation and Communication's decision to develop a limit states format code in metric that included the latest results of research performed for the Ministry. The first edition of the code was issued on a trial basis (OMTC 1979) and was amended several times before being reissued in 1983 after a significant revision process based on field experience in applying the original edition. The code was again substantially revised in the third edition issued in 1991 (OMT 1991).

1979 Bridge Design Code

Provisions for Dynamic Load and Vibration—Section 3 of the 1979 OHBDC (OMTC 1979) contains the provisions for dynamic load allowance and control of vibration. The provisions allow either a static analysis, the results of which are increased by the specified dynamic load allowance, or an approved dynamic analysis to establish dynamic load effects. The dynamic load allowance is applied to all superstructures, including the sidewalks, the substructure components that transmit dynamic loads, and buried components. Excluded are the footings and footing piles. The live loads are (a) a design truck or (b) a lane load that includes a fraction of the design truck load. The DLA is applied to both loads.

The DLA may be specified as a constant or may vary as a function of superstructure fundamental frequency, depending on the component. For instance, the DLA for deck slabs is at least 0.40 if the design is governed by a single or dual axle unit. The DLA is 0.35 for floor beams supporting deck slabs and other beams or slabs, provided their spans are less than 12 m (40 ft). For main longitudinal components of the superstructure and for those components that support these elements, the DLA is based on the first flexural frequency of vibration of the superstructure component. The relationship used is shown in Figure 30.

When more than one design lane is loaded, or if the components are wooden, then the DLA is allowed to be reduced. For multilane loadings the reduction factors are 0.7 for two lanes, 0.6 for three lanes, and 0.5 for four or more lanes. The reason for allowing such reductions is that multiple vehicles do not respond completely in phase with one another, and thus the dynamic effects induced in the structure are less than those

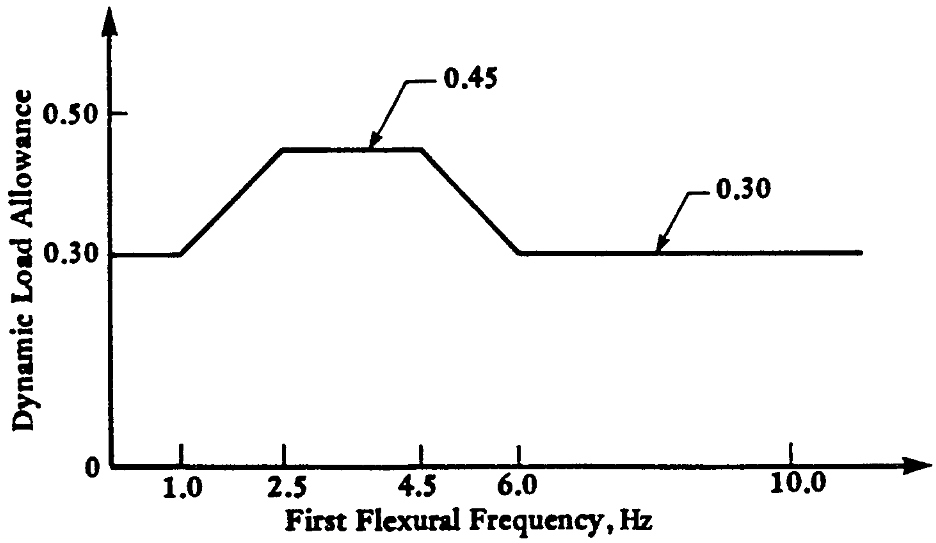

FIGURE 30 1979 OHBDC dynamic load allowance (from OMTC 1979).

induced by a single vehicle. For wooden components, a factor of 0.7 is applied in recognition of the high material damping of wood.

When speeds are restricted on bridges, a reduction is also allowed. The factors are 0.5 for a maximum speed of 25 km/h (15 mph) and 0.3 for a maximum of 10 km/h (6 mph). No interpolation is allowed. This provision recognizes the effect that lower speeds have on the dynamic response induced.

The OHBDC requires that approach slabs of 6-m (20-ft) length be used on all paved roads. This helps to reduce vehicle vertical vibration as the result of uneven approach conditions. Thus the dynamic loading effects induced in the structure will likewise be less than if the vehicle is vibrating vertically when it enters the bridge.

The OHBDC uses deflection limits to control vibrations due to live load. The deflection limits are a function of the first flexural frequency of the superstructure as shown in Figure 31, rather than the length of the span as in the AASHTO *Standard Specifications*. Additionally, for pedestrian bridges, limits of acceleration rather than deflection are given for the control of vibration. These limits are also given as a function of vibration frequency of the superstructure as seen in Figure 32.

Background of the 1979 OHBDC Dynamic Load and Vibration Provisions—The departure from the AASHTO methodology stemmed from the indication that important parameters known to affect dynamic response of bridges, particularly continuous and longer simple span bridges, were not being included by the design codes of the day (Csagoly and Dorton 1978). Further, the span-to-depth ratios, which had railway bridge design as their ancestry, were considered outdated, and better controls on vibration and deflection were available.

The OMTC had performed several field studies aimed at determining the dynamic response of bridges under actual loading and in 1973 (Csagoly and Dorton 1973) included increased impact fractions into their proposed bridge design loading. The tests indicated that when a bridge's natural periods of vibrations coincided with trucks' "bounce and pitch" natural frequencies, then a condition of quasi-resonance occurs

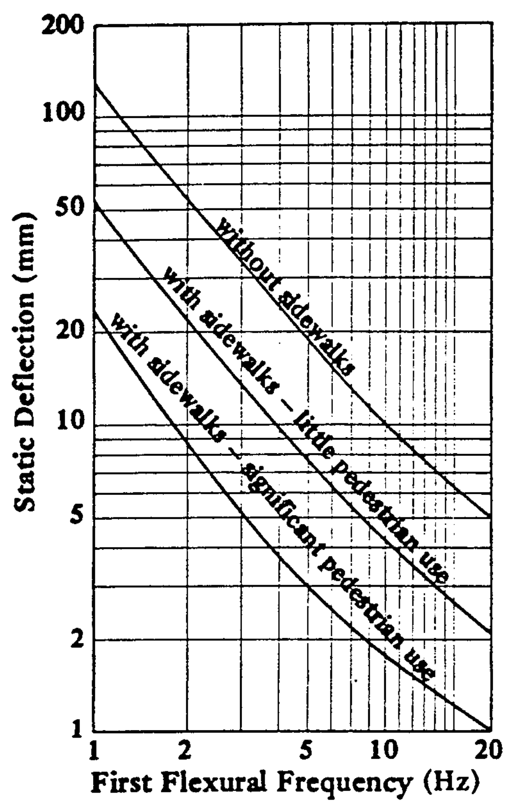

FIGURE 31 1979 OHBDC deflection limits for bridges (from OMTC 1979).

and the actual dynamic amplification may be significantly larger than that indicated by the AASHTO formula. In fact field results from Ontario, even as far back as 1956 (Csagoly and Dorton 1978), indicated that impact fractions could substantially exceed the AASHTO values. Results of the 1956 tests and of the early 1970s OMTC tests are shown in Figures 33 and 34, respectively, taken from the 1979 OHBDC Commentary. These clearly indicate that, in the frequency range

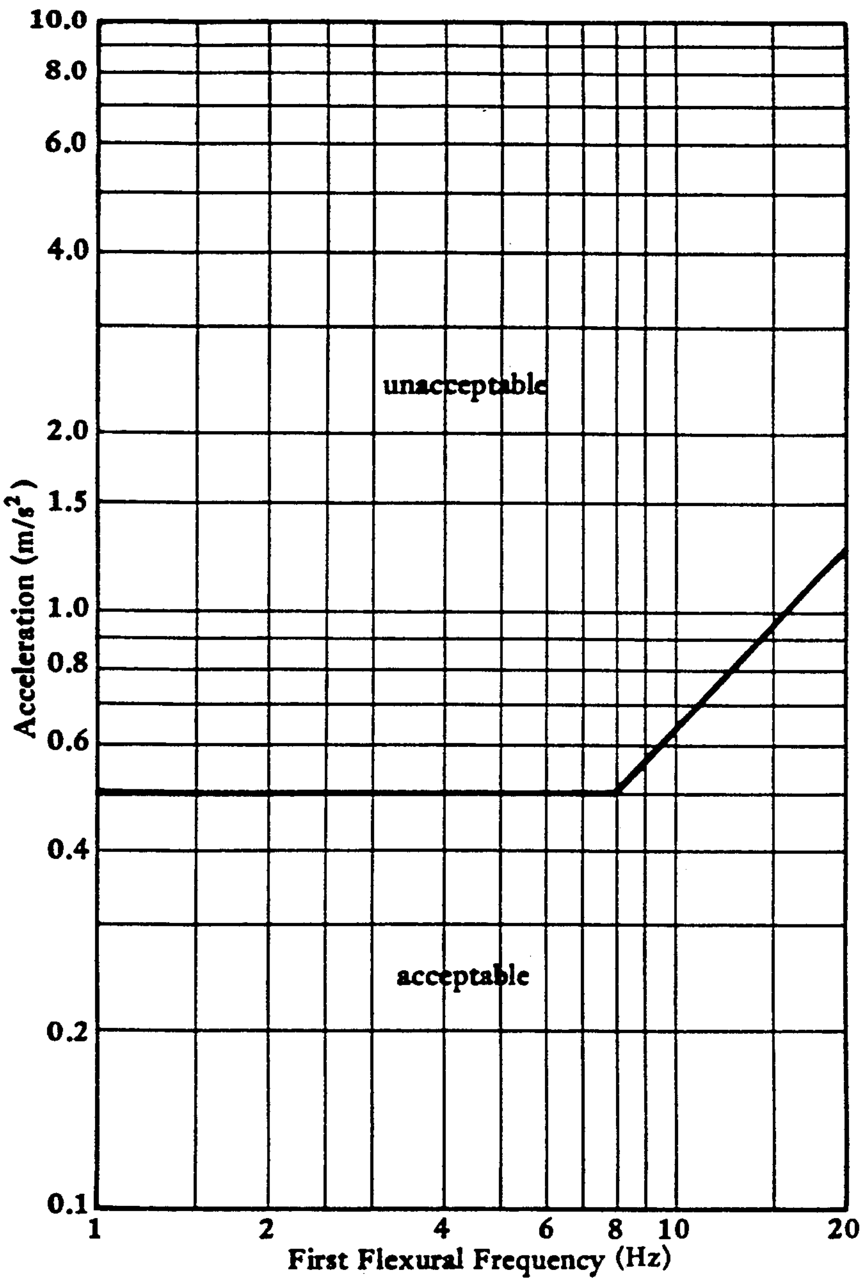

FIGURE 32 1979 OHBDC acceleration limits for pedestrian bridges (from OMTC 1979).

from 2 to 5 Hz, dynamic amplification may exceed the AASHTO maximum of 0.30. The writers of the 1979 OHBDC recognized that making the DLA a function of the first flexural frequency of vibration would introduce an iteration into the design process, and therefore guidance was provided in the Commentary for assuming initial DLA values based on span length and whether the span was continuous or not.

For the control of vibrations, the OHBDC recognizes that limiting human perception to the vertical vibration of bridges is a primary objective. Historically, quasi-empirical deflection limits have been used to control vibrations. However, the 1979 OHBDC provisions are based on limiting acceleration, which is a key parameter for human perception of motion. The criteria in the provisions are given in terms of deflections that were converted from acceleration for ease of calculation.

1983 Bridge Design Code

Provisions for Dynamic Load Allowance—In the 1983 OHBDC (OMTC 1983), the DLA provisions were included within the Live Load Section 2-4.3, and several changes were made from the 1979 OHBDC. The provision for DLA to vary with flexural frequency was amended to apply for spans greater than 22 m (72 ft) and for more than one axle loading. Further, the magnitude of the DLA changed to that shown in

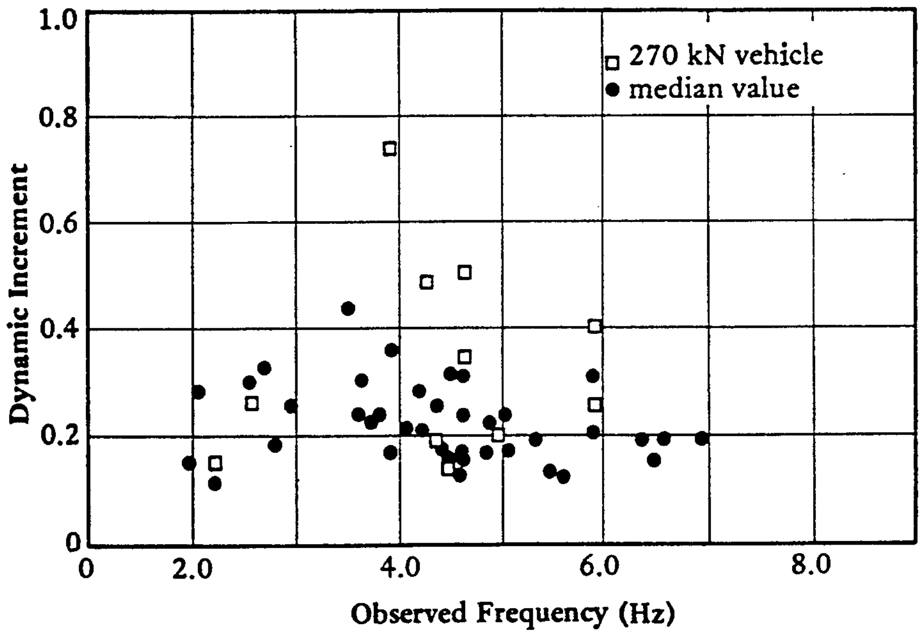

FIGURE 33 Dynamic increment for 1950s Queen's University testing (from OMTC 1979).

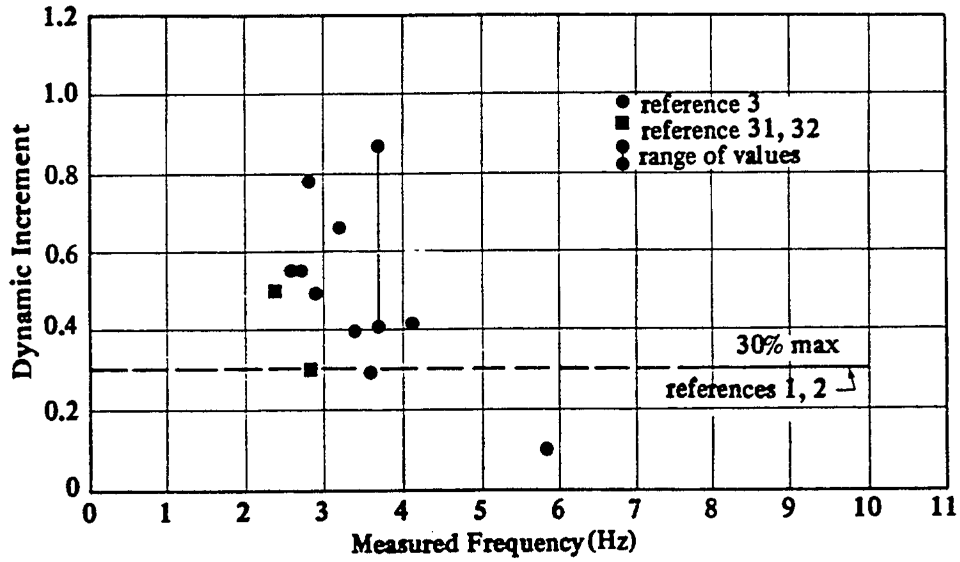

FIGURE 34 Dynamic increment for 1970s Ontario testing (from OMTC 1979).

Figure 35. As can be seen from the figure, the DLA for all frequencies was reduced. A further reduction was affected by specifying a 0.10 DLA for the uniform load portion of the design live load. For spans less than 22 m (72 ft) and for transverse members and for more than a single axle or wheel, the basic DLA became 0.30, instead of 0.35 in the 1979 code.

The multilane modification for DLA was eliminated. However, the multilane modifying factors for the static plus dynamic loads were reduced by 0.05 for nearly every category and thus partially offset the elimination of the reduction on the dynamic load alone.

Approach slabs, which are 6.0 m (20 ft) long, are still required for paved roads.

Provisions for Deflection and Vibration Control—The format of the deflection and vibration control provisions remained essentially the same in the 1983 OHBDC. However, the deflection limits were tightened by reducing the allowable static deflection as seen in Figure 36. The acceleration limits for pedestrian bridges were simplified to a single linear relationship as seen in Figure 37.

Background for the 1983 Dynamic Load Allowance and Deflection Control Provisions

Additional testing was conducted by the OMTC between the publication of the first and second editions of the OHBDC

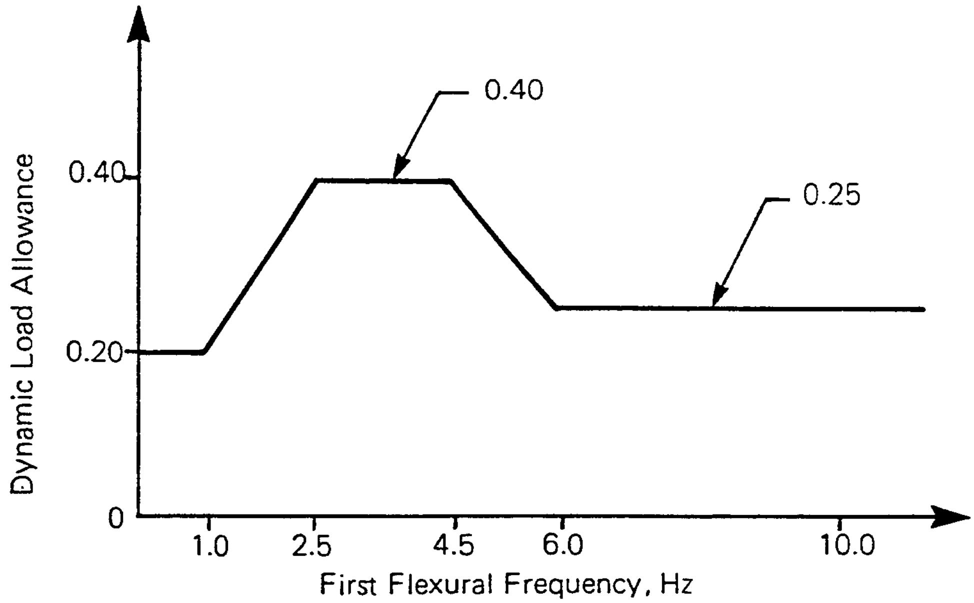

FIGURE 35 1983 OHBDC dynamic load allowance (from OMTC 1983).

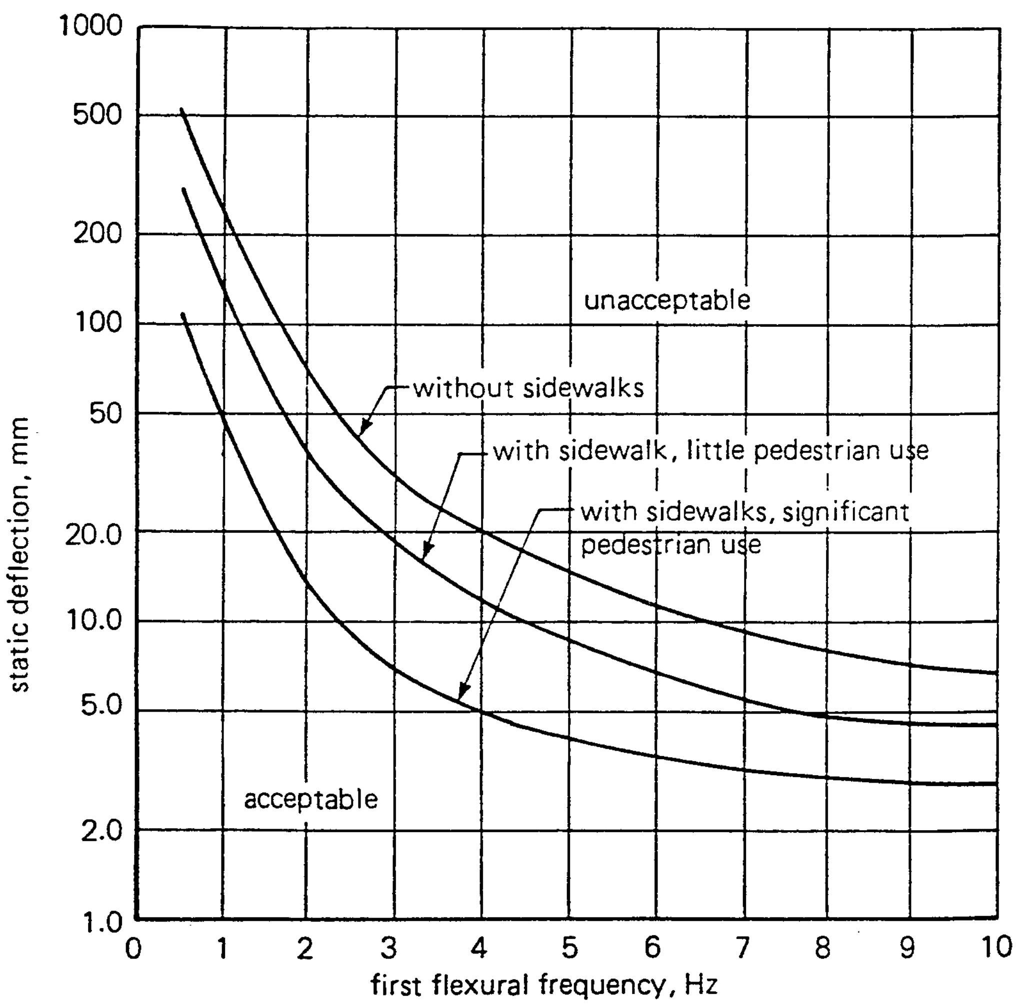

FIGURE 36 1983 OHBDC deflection limits for serviceability (from OMTC 1983).

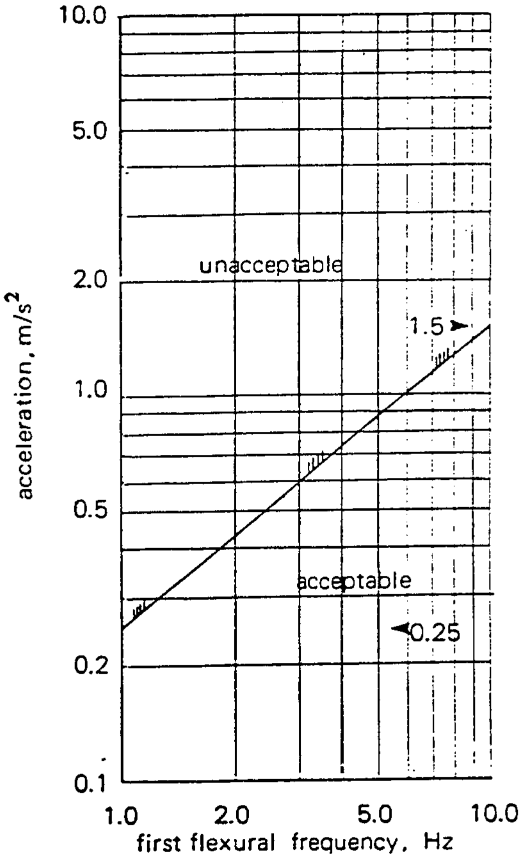

FIGURE 37 1983 OHBDC accleration limits for pedestrian serviceability (from OMTC 1983).

(Billing and Green 1984). Additionally, the statistics of the dynamic amplification factors were re-evaluated as described by Billing and Green (1984). The re-evaluation resulted in the reduction of the DLA for those bridges where the allowance was a function of the flexural frequency.

1991 Bridge Design Code

Provisions for Dynamic Load Allowance—The DLA provisions were substantially simplified in the 1991 OHBDC (OMT 1991) by the elimination of the frequency dependence of the DLA. Instead the DLA was represented as a function of the number of axles considered in the span. The relationship for this is seen in Table 6. Otherwise, the provisions remained unchanged in that a DLA of 0.10 is applied to the uniform load portion of the live load, the DLA for soil-steel structures is 0.40, the DLA may be reduced using a factor of 0.70 if the structure is wooden, and the DLA is not applied to the footings, soil, or footing piles.

The modification factors for multilane loading, which apply to both the live load and its DLA, remained unchanged from the 1983 OHBDC.

Approach slabs, which are 6.0 m (20 ft) long, are required for roads paved with asphaltic concrete wearing surfaces.

TABLE 6

DYNAMIC LOAD ALLOWANCE AS A FUNCTION OF NUMBER OF AXLES 1991 OHBDC (OMT 1991)

Number of Axles	Dynamic Load Allowance
1	0.40
2	0.30
3 or more	0.25

Provisions for Deflection and Vibration Control—The limits for deflections remained unchanged and are thus the same as shown in Figure 36. For pedestrian bridges, the method specified in the 1983 OHBDC was changed to a recommended method, and is thus only outlined in the Commentary.

Background for the 1991 Dynamic Load Allowance and Deflection Control Provisions—The Commentary to the 1991 OHBDC outlines the reasoning for the changes made to the DLA provisions.

It was believed that the basic data on which the OHBDC DLA provisions had been based was sound. However, some reductions in the DLA were believed supportable, thus the DLA values were lowered. A key issue was that of calibration of the code values, specifically what DLA should be used with the design vehicle, which is quite heavy relative to normal loads. It is well known that the dynamic increment or amplification factor goes down as vehicle weight goes up (Billing and Green 1984; Hwang and Nowak 1991; Nowak et al. 1990). Since the OHBDC design truck is 740 kN (166 kips), which is significantly heavier than the trucks that induced the observed amplification factors in the field, it was believed that the DLA could be reduced for the heavy OHBDC design load. In fact, it was determined that a DLA of 0.20 would be supportable, even in the frequency sensitive range of 2 to 5 Hz. However, there was reluctance to drop the DLA below 0.25; that then became the new, 1991, DLA for three or more axles. Consideration of fewer axles led to the 0.40 and 0.30 values for one and two axles, respectively.

It is emphasized in the Commentary that for vehicles that are significantly lighter than the 740-kN (166-kip) OHBDC truck, the DLA should be higher than that given by the 1991 provisions. In the 2 to 5 Hz flexural frequency range, where the previous editions of the OHBDC gave high DLAs, the underestimation of DLA for lighter loads could be significant due to quasi-resonance that is developed between the vehicle and the bridge superstructure. This reasoning means that a comparison between the AASHTO and the Ontario DLA provisions must recognize the differences between the design truck loads.

The values of DLA presented represent surface conditions that are generally smooth and reasonably free from undulations. It is well known that adverse surface roughness can lead to dynamic amplifications that are significantly in excess of the code-specified values. For this reason, the OHBDC Commentary recommends that the DLA be increased from 0.40 to 0.50 in locations adjacent to expansion joints that may not be adequately flush with the roadway. This increase is recommended over one-tenth of the span.

TABLE 7
1996 JAPAN ROAD ASSOCIATION IMPACT COEFFICIENTS (from JRA 1996)

Kind of Bridge	Impact Fraction	Kind of Loadings to be Applied
Steel bridge	$i = \frac{20}{50 + L}$	T-loading and L-Loading
Reinforced concrete bridge	$i = \frac{20}{50 + L}$	T-Loading
	$i = \frac{7}{20 + L}$	L-Loading
Prestressed concrete bridge	$i = \frac{20}{50 + L}$	T-Loading
	$i = \frac{10}{25 + L}$	L-Loading

L = span length (m).

BRITISH STANDARDS INSTITUTE

The British Standards Institute's BS 5400 *Steel, Concrete and Composite Bridges Part 2. Specification for Loads* (BSI 1996) gives two highway bridge live loads, the HA and HB loads, which are meant to cover normal traffic loads and "abnormal" loads, respectively. For both live loads, a 25 percent allowance for impact or dynamic effects is included in the load itself. Appendix A of the BS 5400 document describes the basis for the HA and HB loadings, and the Appendix notes that the loading and the impact allowances used have been found to give "satisfactory correspondence in behaviour."

JAPAN ROAD ASSOCIATION

The Japan Road Association's *Specifications for Highway Bridges* (JRA 1996) gives impact provisions for all bridge types in Section 2.1.4 of Part I Common Specifications. The Japanese *Specifications* treat impact in a similar fashion to the AASHTO *Standard Specifications* in that impact is a function of span length. The impact fraction is also dependent on the type of bridge (steel, concrete, or prestressed concrete) and the type of loading (truck (T) or lane (L)) as shown in Table 7.

It is seen in Table 7 that the form of the relationship is identical to the AASHTO *Standard Specifications*, with the span length given in the denominator added to a constant. It can also be seen that the impact fraction is the same for all types of bridges for the truck loading, but it is different for all bridges for the lane loading. In general, the lane loading impacts are less than that for trucks, and this is indicative of the lower dynamic amplification that occurs when multiple vehicles are on the span. Furthermore, it can be seen that for the types of bridges with the highest material damping (concrete), the impact fraction is lowest, and the fraction increases as the damping goes down. Steel has the highest impact fraction for the lane loading.

The Japanese *Specifications* excludes sidewalk live loads and the main cables and stiffening girders of suspension bridges due to low likelihood of significant interaction of the loads and the bridge. Similar to the AASHTO *Standard Specifications*, the increased forces due to impact are not considered in the design of footings. The Japanese *Specifications* provides rather detailed definitions of the span length to use in the calculation of the impact fraction.

Finally, the values of impact fraction for the various bridge and loading types are compared in the Japanese *Specifications* against the AASHTO, the German DIN 1072, and the French Fascicule 61, Titre II du CPC specifications. For comparison, this graphic is given in Figure 38.

AMERICAN RAILWAY ENGINEERING ASSOCIATION (AREA)

The AREA *Manual for Railway Engineering* (AREA 1996) includes impact separately for the design of concrete and steel bridges. Additionally, the impact provisions for evaluation of existing bridges is different from the design provisions. As with other design specifications, the impact is added as a fraction of the static effect.

Concrete Bridges

For concrete bridges, the axle loads are increased by a percentage equal to:

$$I = 100\, L/(L + D) \tag{11}$$

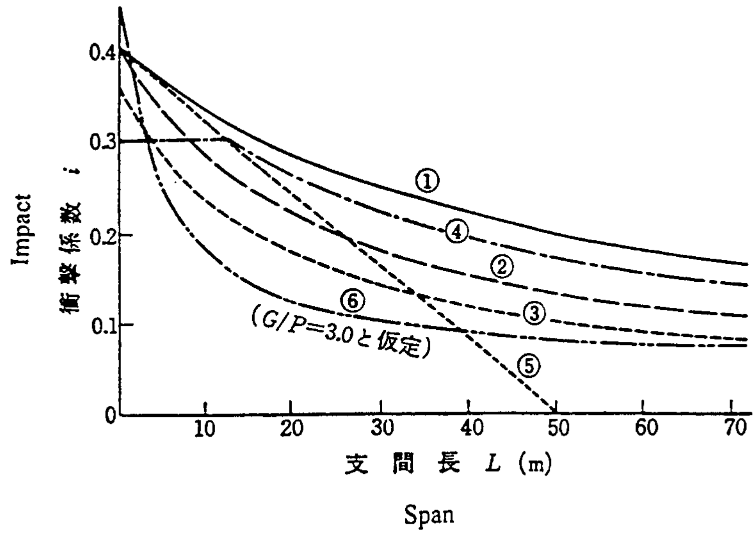

FIGURE 38 Japansese impact relationship compared to those of other countries (from JRA 1996).

1 = Japan—steel, concrete, and prestressed concrete bridges for truck loading
2 = Japan—prestressed concrete bridge for lane loading
3 = Japan—reinforced concrete bridge for lane loading
4 = AASHTO—all types
5 = Germany—all types
6 = France—all types with dead to live load (G/P) of 0.3.

where L is the total live load on the member under consideration and D is the dead load of the same. The impact percentage does not need to exceed 80 percent for steam locomotive impact nor 60 percent for diesel locomotives.

Steel Bridges

For steel bridges the impact percentage is distinguished by whether "hammer blow" is present. Hammer blow is the periodic forces applied to the structure from steam locomotives due to unbalanced reciprocating drive mechanisms. Thus, rolling equipment without hammer blow includes diesel and electric locomotives and tenders.

The percentage of impact for equipment without hammer blow is

$$I = RE + 40 - 3\,L^2/1600 \qquad L < 80 \text{ feet*} \qquad (12)$$

and

$$I = RE + 16 + 600/(L - 30) \qquad L > 80 \text{ feet*} \qquad (13)$$

where RE is the rocking effect of the locomotive and is taken as 10 percent of the axle load or 20 percent of the wheel load, and L is the span.

Where hammer blow is present, the percentage of impact is:

$$I = RE + 60 - L^2/500 \quad L < 100 \text{ feet*} \qquad (14)$$

and

$$I = RE + 10 + 1800/(L - 40) \quad L > 100 \text{ feet*} \qquad (15)$$

For comparison, these two relationships for impact, as well as the AASHTO *Standard Specifications'* impact equation, are plotted in Figure 39. In the figure, the addition to the AREA expressions for the rocking effect, RE, has been omitted.

For trusses, where hammer blow is present, the percentage impact is:

$$I = RE + 15 + 4000/(L + 25) \qquad (16)$$

If more than one track loads a particular member, then reduction factors are specified to recognize the fact that multiple dynamic loads do not induce the same maximum dynamic effect as a single load.

Further, the provisions recognize that for ballasted deck structures, in comparison to open deck structures, additional damping is present that reduces the effects of impact. The AREA *Manual* allows a 10 percent decrease in impact for such structures.

Timber Bridges and Trestles

The AREA *Manual* includes dynamic loading effects indirectly as described in Section 2.5.5.6:

> The dynamic increment of load due to the effects of speed, roll and track irregularities is not well established for timber structures. Its total effect is estimated to be less than the increased strength of timber for the short cumulative duration of loading to which railroad bridges are subjected in service, and is taken into consideration in the derivation of the allowable working stress for design.

*SI units are not provided due to constant values in the equation.

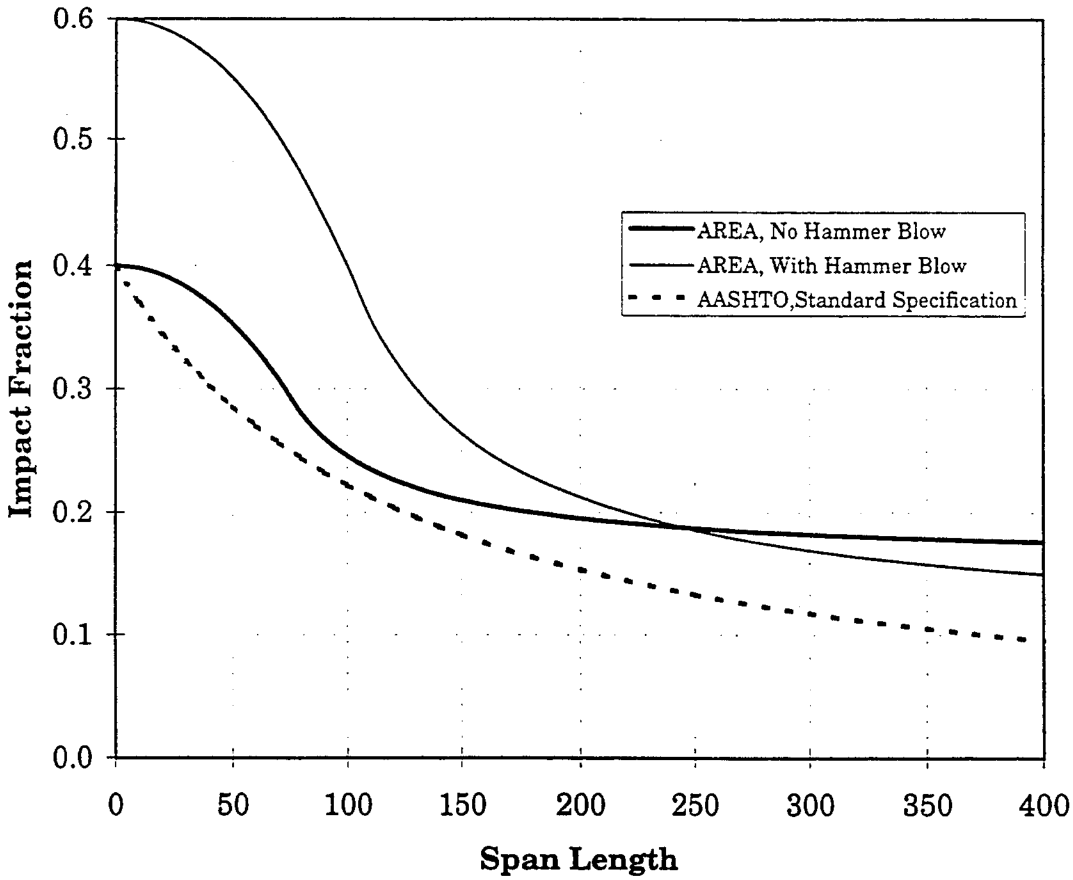

FIGURE 39 AREA impact for steel bridges compared with those in AASHTO (from AREA 1996).

Existing Bridge Evaluation

For evaluation, the AREA *Manual* allows reductions in the second and third terms of the values given for steel structures above if the speed of the locomotive is restricted. Also for trusses loaded with locomotives with hammer blow, reductions are allowed to the second and third terms of the equations provided that revolutions per second of the locomotive drivers do not equal that of the flexural vibration frequency of the structure. This recognizes the resonant condition that will develop should the two frequencies be equal or nearly so.

TRANSIT GUIDEWAY CONSTRUCTION

The design of concrete transit guideway structures is covered by the 1996 ACI 358 Report, "Analysis and Design of Reinforced and Prestressed Concrete Guideway Structures" (ACI 1996), and for items that are not addressed in this set of recommendations, highway and railway design specifications are applied. This report contains essentially an LRFD-based set of recommendations in that they have been developed to yield roughly consistent reliability indices, much like the AASHTO *LRFD Specifications*. However, the reliability has been set slightly higher than the bridge design values to compensate for the higher consequences of failure of a public transit system.

The provisions for vertical impact are given in Section 3.3.1.2 of the report, and the impact factor, I, is shown in Table 8, where the vehicle crossing frequency is given by:

$$VCF = \text{vehicle speed/span length} \tag{17}$$

and the first flexural frequency (f_1) for a simple span with uniform weight is:

$$f_1 = (\pi / 2\, L^2)\,(E_c\, I_g / M)^{1/2} \tag{18}$$

In the equation, L is the span length, E_c is Young's modulus, I_g is the gross section moment of inertia, and M is the mass per unit length of all sustained vertical loads. As seen in the table, the value used for impact depends on the smoothness of the rail and the vehicle tire type, as well as the continuity of the superstructure. The impact factor is to be applied to vertical live load effects in all members except footings or piles, and alternative impact factors may be used if substantiated by test data or by approved dynamic analyses.

In addition to impact factors for vertical live load effects, the ACI 358 report provides guidance for dealing with other transient load effects, such as centrifugal, longitudinal, and

TABLE 8

DYNAMIC LOAD ALLOWANCE FOR GUIDEWAY STRUCTURES (from ACI 1996)

Stucture Types	Rubber-Tired and Continuously Welded Rail	Jointed Rail
Simple-span structures,		
$I = \dfrac{VCF}{f_1} - 0.1$	≥ 0.10	≥ 0.30
Continuous-span structures,		
$I = \dfrac{VCF}{2f_1} - 0.1$	≥ 0.10	≥ 0.30

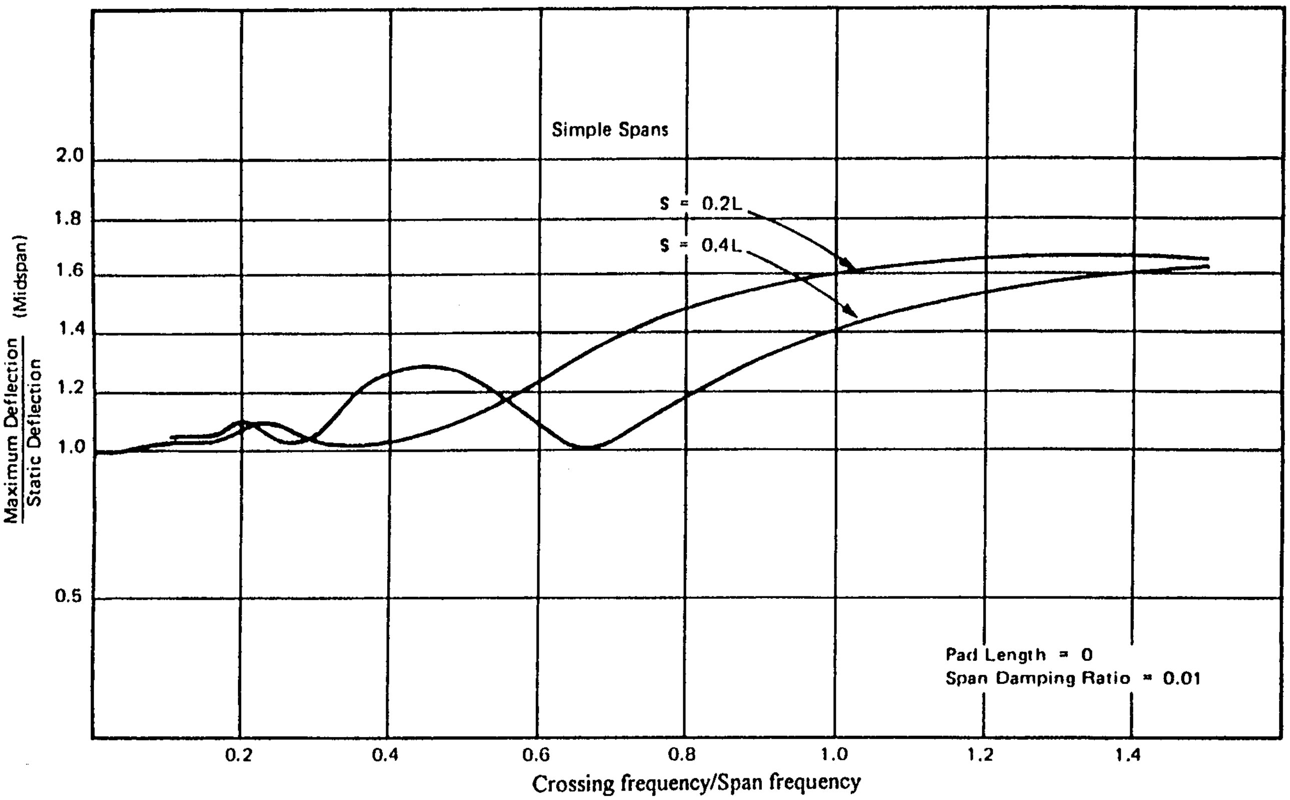

FIGURE 40 Deflection amplification for different crossing frequencies and axle spacings (from AISI 1974).

'hunting' forces. Hunting (or nosing) forces are caused by the structure 'guiding' the vehicle as it progresses. The provisions given in the ACI 358 report provide general guidance for designers. However, dynamic response is a function of the vehicle suspension and therefore detailed consideration of vehicle-structure dynamic interaction must consider the vehicle, itself. This may require close coordination between the guideway designer and the vehicle supplier.

The values of impact factors listed for concrete structures are similar in form and in magnitude to those described for steel transit structures in AISI (1974). In fact that document, which was published by AISI in 1974, gives some background discussion on the issue of impact, and consistent with the thinking and analytical studies of the day, impact was considered strongly dependent on the ratio of the crossing frequency to natural frequency of the bridge. This is especially so for the case of point loads with little suspension moving over a span, and is probably appropriate for transit structures. However, for heavy trucks with suspension systems that allow oscillation modes of the truck itself, the crossing frequency does not seem to be of primary importance. The analytically based impact relations given in the AISI document are shown in Figure 40. This gives impact as a function of crossing and span frequencies, CF/SF, for simple spans. The two curves are for different axle spacings, s, as a function of span length, L. The increase in impact as the ratio of frequencies increases is seen, and a quasi-resonance condition would be approached for CF/SF ratios of 2.0. This is due to the fact that loads moving over a span only induce deformations that correspond to one-half of the vibration cycle.

CHAPTER FIVE

STATE OF PRACTICE AND REPORTED FIELD PROBLEMS

INTRODUCTION

This chapter presents a summary of current practices in accounting for vehicular dynamic effects in the design of new bridges and when load rating existing bridges. Current practices were established based on the responses from a questionnaire distributed to state and provincial transportation agencies as part of this synthesis development. The questionnaire (Appendix A) requested responses on field problems in existing bridges associated with vehicular dynamic effects, which are summarized in this chapter. Field problems reported in the literature are also briefly discussed.

CURRENT DESIGN AND EVALUATION PRACTICES

Responses from the state and provincial agencies returning the questionnaire on current design and evaluation practices are summarized in Table 9. Information was requested on current procedures for designing for vehicular dynamic load effects in new bridges and for evaluating existing bridges for load rating and permit loads. The questionnaire was sent to all 50 states plus the District of Columbia and Puerto Rico in the United States, and 41 responses were obtained. The questionnaire was sent to all 10 provinces and the two territories in Canada, and three responses were obtained.

Designing for Dynamic Load Effects in New Bridges

Of the 41 U.S. responses, the current AASHTO *Standard Specifications for Highway Bridges* is used in 32 states, without modification, to account for vehicular dynamic load effects in the design of new bridges. The AASHTO *LRFD Design Specifications* are used in only one state (Hawaii), and eight states report that they are "piloting" the *LRFD Specifications* along with using the *Standard Specifications*. Hawaii reported complications in using the *LRFD Specifications* in regard to "distributing the loads according to the LRFD." Three states report that they are in the process of changing from using the *Standard Specifications* to the *LRFD Specifications*.

All three provincial agencies responding to the questionnaire indicate that they are using the 1988 CAN/CSA-S6 *Design of Highway Bridges*. Two of the three provincial agencies responding reported significant complications in accounting for dynamic load effects due to "the need to calculate the natural frequency of the structure." The Canadian respondents note that a new Canadian design code will soon be out.

Maine reported sponsoring research investigating load values in the *LRFD Specifications*. The focus of this research is to investigate the live load lateral distribution factors in the *LRFD Specifications* and to determine if the *LRFD Specifications* result in decreased superstructure capacity in comparison to that obtained using the *Standard Specifications*. North Carolina reported on continuing research efforts to develop improved expansion joint details. This research investigated the use of polyethylene copolymer foam seals as watertight bridge deck joints. Details on these joint studies can be found in reports by Stanley (1991, 1995). Virginia reported investigating the combined effects of increased girder spacing and dynamic loads on reinforced concrete decks. The Wyoming Department of Transportation indicated that they have instrumented several bridges with strain transducers to examine the effects on the bridges of over-load vehicles.

Evaluation of Load Effects in Existing Bridges

From the questionnaire responses, the same criteria for new bridge design are used to account for vehicular dynamic effects when evaluating load ratings for existing bridges in 18 states and three provinces. The AASHTO *Load Rating Guide Specification* is used by 14 states and the AASHTO *Manual for Condition Evaluation of Bridges* is used by one state. A combination of the three specifications is used in eight states.

For overload permit loads, 20 states and all three provinces indicated that special provisions for vehicular dynamic load effects are used. Vehicle speed is restricted and the impact loading considered is reduced or eliminated. Twelve states and two provinces reported reduced dynamic loads of unspecified or case-by-case values for reduced vehicle speeds. One state reported using half of the normal impact for speeds $\leq$ 24 kph (15 mph), two states use an impact factor of 10 percent for speeds $\leq$ 16 kph (10 mph), one state uses 10 percent for speeds $\leq$ 8 kph (5 mph), and one province reported using an impact factor of 5 percent for reduced, but unspecified, speeds. Four states report using no impact factors for permit vehicles when the vehicle speed is restricted to $\leq$ 8 kph (5 mph).

FIELD PROBLEMS IN EXISTING BRIDGES

Survey Responses

Field problems provided by the responses from the questionnaire are summarized in Table 10. Eleven agencies reported that they have not experienced any problems in existing bridges attributable to vehicular dynamic load effects. Possible

TABLE 9

CURRENT DESIGN AND EVALUATION PRACTICES

	Design				Load Rating Existing Bridges		
Agency	Specifications Used	Complications in the Design Process	Changes Considered	Currently Involved in Research	Evaluation Method	Application Differences	Changes Considered
Alaska	Standard	N	N	N	Same as for new bridge design	N	N
Arizona	Standard	N	N	N	AASHTO Load Rating Guide Specification	Y	N
Arkansas	Standard & LRFD	N	N	N	Same as for new bridge design	N	N
California	Standard	N	Y	N	Same as for new bridge design	N	N
Colorado	Standard	N	Y	N	Same as for new bridge design	Y	N
District of Columbia	Standard	N	N	N	AASHTO Load Rating Guide Specification	N	N
Florida	Standard	N	N	N	AASHTO Load Rating Guide Specification	Y	N
Georgia	Standard	N	N	N	Same as for new bridge design	N	N
Hawaii	LRFD	Y	N	N	AASHTO Load Rating Guide Specification	N	N
Idaho	Standard	N	N	N	Same as for new bridge design	Y	N
Illinois	Standard	N	N	N	AASHTO Load Rating Guide Specification and same as for new bridge design	Y	N
Indiana	Standard	N	N	N	Same as for new bridge design	Y	N
Iowa	Standard	N	N	N	AASHTO Load Rating Guide Specification	Y	N
Kansas	Standard	N	N	N	AASHTO Load Rating Guide Specification	Y	N
Kentucky	Standard	N	N	N	AASHTO Load Rating Guide Specification and same as for new bridge design	N	N
Louisana	Standard & LRFD	N	N	N	Same as for new bridge design	Y	N
Maine	Standard & LRFD	N	N	Y	AASHTO Load Rating Guide Specification	Y	N
Massachusetts	Standard	N	N	Y	Same as for new bridge design	Y	N
Michigan	Standard	N	N	Y	Same as for new bridge design	N	N
Minnesota	Standard	N	N	N	AASHTO Load Rating Guide Specification and same as for new bridge design	N	N
Mississippi	Standard	N	N	N	AASHTO Load Rating Guide Specification	N	N
Missouri	Standard	N	Y	N	Same as for new bridge design	N	N
Montana	Standard & LRFD	N	N	N	AASHTO Load Rating Guide Specification and same as for new bridge design	Y	N
Nebraska	Standard	N	N	N	AASHTO Manual for Condition Evaluation of Bridges	Y	N
Nevada	Standard	N	N	N	Same as for new bridge design	N	N
New Jersey	Standard	N	N	N	AASHTO Load Rating Guide Specification and AASHTO Manual for Condition Evaluation of Bridges	N	N
New Mexico	Standard	N	N	N	AASHTO Load Rating Guide Specification	N	N
New York	Standard	N	N	N	Same as for new bridge design	Y	N
North Carolina	Standard	N	N	Y	Same as for new bridge design	Y	N
North Dakota	Standard	N	N	N	Same as for new bridge design	N	N
Oklahoma	Standard & LRFD	N	N	Y	Same as for new bridge design	N	N
Oregon	Standard	N	N	N	AASHTO Load Rating Guide Specification	N	N
Pennsylvania	Standard & LRFD	N	N	N	AASHTO Load Rating Guide Specification and same as for new bridge design	Y	N

TABLE 9 (Continued)

	Design				Load Rating Existing Bridges		
Agency	Specifications Used	Complications in the Design Process	Changes Considered	Currently Involved in Research	Evaluation Method	Application Differences	Changes Considered
Rhode Island	Standard	N	N	N	AASHTO Load Rating Guide Specification and same as for new bridge design	N	N
South Carolina	Standard	N	N	N	AASHTO Load Rating Guide Specification	Y	N
Tennessee	Standard & LRFD	N	N	N	Same as for new bridge design	Y	N
Texas	Standard	N	N	Y	AASHTO Load Rating Guide Specification	N	N
Virginia	Standard	N	N	Y	AASHTO Load Rating Guide Specification	Y	N
Washington	Standard & LRFD	N	N	N	AASHTO Load Rating Guide Specification	N	N
Wisconsin	Standard	N	N	N	Same as for new bridge design	N	N
Wyoming	Standard	N	N	Y	AASHTO Load Rating Guide Specification and same as for new bridge design	Y	N
CANADA							
New Brunswick	CAN/CSA-S6-88	Y	Y	N	Same as for new bridge design	Y	Y
Newfoundland	CAN/CSA-S6-88	N	Y	N	Same as for new bridge design	Y	N
Saskatchewan	CAN/CSA-S6-88	Y	Y	N	CAN/CSA-S6-88	Y	Y

TABLE 10

REPORTED FIELD PROBLEMS IN EXISTING BRIDGES

	Field Problems in Existing Bridges							
Agency	Problems Experienced	Extent of Problems	No. of Bridges with Problems	Percent of Bridge Inventory	Location of Problems	Common Problems and/or Specific Bridge Types	Associated Problems	Revisions in Inspection, Maintenance or Design Procedure
Alaska	N							
Arizona	Possibly	Minor			Expansion joints Concrete decks on steel girders	Approach slab settlement Deck joint failure	Serviceability	Implemented program to address the approach slab settlement problem
Arkansas	Possibly	Minor			Expansion joints	Sliding plate expansion joints-connection	Failure	Use of LRFD Specification for Design
California	Possibly	Moderate			Main girders Connections Expansion joints	Out of plane bending of steel girders Aluminum joint seals	Serviceability (girders) and failures (joint seals)	No longer use aluminum joint seal assemblies. Connect diaphragms to webs of steel girders with bolted connections
Colorado	Y	Minor			Expansion joints Slabs near expansion joints	Cracking of deck slab	Serviceability	None
District of Columbia	Possibly	Minor			Connections	None	Serviceability	None
Florida	Possibly	Minor			Deck Parapets	Light superstructures Long steel spans	Serviceability	None

TABLE 10 (Continued)

Agency	Problems Experienced	Extent of Problems	No. of Bridges with Problems	Percent of Bridge Inventory	Location of Problems	Common Problems and/or Specific Bridge Types	Associated Problems	Revisions in Inspection, Maintenance or Design Procedure
	Field Problems in Existing Bridges							
Georgia	Possibly	Minor	15	1.00	Main girders Connections Expansion joints	Long span steel bridges	Serviceability	Increased inspection frequency for fatigue/impact related problems
Hawaii	N							
Idaho	Y	Moderate			Bearings Expansion joints		Serviceability and failures	Improved expansion joint details
Illinois	N							
Indiana	Possibly	Moderate	50 ±	1.00	Main girders Bridge deck Bridge deck edge	Overlay failure	Serviceability	Removal of expansion joint wherever possible as part of bridge rehabilitation
Iowa	N							
Kansas	Y	Moderate	250	5.00	Main girders Connections Expansion joints	Areas of out of plane bending of steel members	Serviceability and failures	Use more bolted connections
Kentucky	N							
Louisiana	Possibly	Minor			None		Serviceability	None
Maine	Yes	Minor	100	3.00	Expansion joints Decks	Modular joints Approach slabs	Serviceability	None
Massachusetts	Possibly				Expansion joints	Cracked or broken concrete parapets or deck ends	Serviceability	None
Michigan	Possibly	Minor		5.00	Bearings Connections	None	Serviceability	Improve elastomeric bearing pad assembly
Minnesota	Y	Moderate			Main girders Connections	Welded girders Connections Cover plates	Serviceability	Increase inspections Use more bolted connections
Mississippi	N							
Missouri	Y	Moderate			Main girders Bearings Connections Expansion joints	Fatigue cracks in plate girders near connections to diaphragms	Serviceability	Eliminated elastomeric expansion devices Changed diaphragm connection to be more fatigue resistant
Montana	Possibly	Minor		<5.00	Expansion joints Lateral bracing	None	Serviceability	None
Nebraska	Y	Moderate	30	1.40	Main girders Expansion joints	Welded plate girders (web cracking caused by out-of-plane bending)	Serviceability	Design for a rigid connection between separator plates and tension flange
Nevada	Y	Minor			Main girders Expansion joints	Armored expansion joints have had concrete and weld failures Skewed steel bridges Bridges with asymmetric cross-bracing	Serviceability	Increased inspection for fatigue-related problems

TABLE 10 (Continued)

Agency	Field Problems in Existing Bridges							
	Problems Experienced	Extent of Problems	No. of Bridges with Problems	Percent of Bridge Inventory	Location of Problems	Common Problems and/or Specific Bridge Types	Associated Problems	Revisions in Inspection, Maintenance or Design Procedure
New Jersey	Y	Moderate	150 ±	1.00	Main girders Connections Expansion joints	Bascule bridges Structural members exhibiting section losses	Serviceability	None
New Mexico	Y	Moderate	250 ±	10.00	Main girders Connections Expansion joints	Expansion joint assemblies Steel bridge girders at diaphragm connections	Serviceability	Revisions considered but none listed
New York	N							
North Carolina	Y	Wide-spread		40.00	Expansion joints	None	Serviceability and failure	No longer use aluminum expansion joints
North Dakota	N							
Oklahoma	Possibly	Minor			Connections Expansion joints Diaphragms	Steel bridges	Serviceability	Increase maintenance
Oregon	Y	Minor			Deck	None	Serviceability	Improved joint and end panel details
Pennsylvania	Possibly	Moderate			Expansion joints	Expansion joints experience distress due to pounding	Serviceability	None
Rhode Island	Possibly	Moderate			Expansion joints Deck Backwalls surrounding the joints	Single or multiple span bridge types composed of a composite steel—concrete superstructure	Serviceability	Use of asphaltic plug joints as a replacement for joint hardware
South Carolina	Y	Moderate			Main girders Connections Expansion joints Precast maintenance slabs		Serviceability	Redesign all precast maintenance bridges Replacement of aluminum modular joints
Tennessee	Possibly	Minor			Expansion joints	None	Serviceability	None
Texas	N							
Virginia	N							
Washington	Y	Minor	3	0.10	Main girders	Steel ties arch bridges 400 ft span ±	Serviceability	Conducting research to determine the effectiveness of repairs made to floor beam

TABLE 10 (Continued)

Agency	Problems Experienced	Extent of Problems	No. of Bridges with Problems	Percent of Bridge Inventory	Location of Problems	Common Problems and/or Specific Bridge Types	Associated Problems	Revisions in Inspection, Maintenance or Design Procedure
	Field Problems in Existing Bridges							
Wisconsin	Possibly	Moderate	400	10.00	Deck	Steel girders with more flexibility than concrete girders	Serviceability	None
Wyoming	Possibly	Minor	30	1.50	Expansion joints	None	Serviceability	None
CANADA								
New Brunswick	Possibly	Moderate			Connections Expansion joints Deck	Steel grid deck sections failing Welds fracture Stringer clip angle connections	Serviceability and failure	Inspection of riveted connections monthly
Newfoundland	Possibly	Moderate	200	25.00	Expansion joints		Serviceability	Improved joint installation Use of snow plow plate at joints
Saskatchewan	N							

field problems attributable to vehicular dynamic loads were reported by 19 agencies, and 14 agencies reported there were field problems. Of the 33 agencies answering yes or possibly, 16 reported the problems to be minor, 15 reported the problems to be moderate, and only one agency reported the problems as widespread (one agency provided no response in regard to the extent of the observed problems). The agency reporting widespread problems identified expansion joints as the only problem area, and the agency estimated that 40 percent of the bridges in its inventory experienced this type of problem.

For those agencies reporting yes or possible field problems, the following bridge components were identified as having experienced damage due to dynamic loads. The number next to the component indicates the number of agencies citing it as experiencing a problem. Note that multiple components were cited by many agencies.

	Agencies
• Expansion joints	22
• Main girders	12
• Connections	12
• Concrete decks	9
• Bearings	3
• Lateral bracing or diaphragms	2
• Expansion joint backwall	1
• Precast maintenance slabs	1
• Parapet walls	1

The observed field problems were reported by 27 agencies as being primarily serviceability problems; five agencies reported serviceability problems and failures, with the failures occurring mainly in expansion joints. Reported passenger discomfort due to bridge vibration was noted by one agency.

Agencies reporting field problems attributable to vehicular dynamic load effects cited approach slab settlements (two agencies), deck joint failures (four agencies) and overlay failures (one agency) as being contributing factors to the observed damage. Sliding plate expansion joints, armored expansion joints, and aluminum joints and joint-seal assemblies were identified as common joint details experiencing damage attributed to dynamic loading. Field problems, mainly in the form of fatigue cracking, were cited in steel bridges 19 times. The reported common problems and specific bridge arrangements associated with field problems in steel bridges are:

Problems	*Agencies*
• Fatigue cracks in welded plate girders, including cover plates	5
• Out-of-plane bending of steel girders, due to symmetrical loading of bridge and resulting differential deflections in the girders, resulting in cracking in the cross-bracing or diaphragm connections	5
• Fatigue cracks in floor beam connections	1
• Fatigue cracking in connections	1
• Weld fractures	1
• Fatigue cracking in steel girder deck sections	1
• Deck cracking with light steel superstructures	1
• Deck cracking with single and multiple simple-span steel girders/concrete deck composite bridges	1

Arrangements	*Agencies*
• Long span steel bridges	4
• Skewed steel bridges	2
• Asymmetric cross-bracing	1
• Long span, tied steel arches	1

In response to the observed field problems, the following changes in inspection, maintenance or design procedures were reported:

Inspection	*Agencies*
• Increased inspection for fatigue-related problems	3
• Inspection of riveted connections monthly	1

Maintenance and Rehabilitation	*Agencies*
• Replacement of aluminum modular joints	1
• Removal of expansion joints wherever possible as part of bridge rehabilitation	1
• Use of asphaltic plug joints as a replacement for joint hardware	1
• Repair approach slab settlements	1

New Design	*Agencies*
• Use of improved joint details	5
• No longer use aluminum joints and joint seal assemblies	2
• Improved end panel joint details	1
• Eliminated elastomeric expansion devices	1
• Use of bolted connections rather than welding agencies	3
• Connect diaphragms to webs of steel girders using bolted connections rather than welding	1
• Design for a rigid connection between separator plates and tension flanges	1

Literature Reports

The number of references documenting damage to bridge structures due to vehicular dynamic loading is relatively few. Most examples of "vehicular" dynamic loads leading to collapse have been associated with pedestrian-induced vibrations, going back as far as the 1833 Brighton Chain Bridge in England and including more recent foot-bridges that have been "bounced" off their bearings (Tilly 1978). However, these examples of damage are irrelevant to modern highway bridges. Tilly (1978) notes that "there are no known cases of (significant) damage to main components due to traffic-induced vibrations although there have been examples of fatigue failure to components, such as cross-bracing."

There is difficulty in attributing fatigue damage observed in bridges to dynamic loads per se, as opposed to just vehicle loading. Correspondingly, there is some confusion and a paucity of discussion of this issue in the literature. King, Csagoly, and Fisher (King et al. 1975) reported on fatigue cracks in haunched wide-flange girders. Field measurements indicated impact factors of 35 percent for the bridge, and the high impact loads were attributed as contributing to the formation and growth of the fatigue cracks.

CHAPTER SIX

CONCLUSIONS

This synthesis presented relevant background and recent information on the dynamic response of bridges under vehicular loading. The synthesis provided a review of basic dynamic principles along with a review of domestic and foreign literature on bridge dynamic response. From this review, the main findings and conclusions on the effects of several parameters on bridge response were summarized. Current code provisions for accounting for dynamic load effects were presented. Results from a questionnaire sent to transportation agencies in the United States and Canada provided information on current practice for the design and load evaluation of existing bridges and identified field problems associated with vehicular dynamic load effects.

The dynamic response of a bridge to a crossing vehicle is a complex problem that requires fairly complex treatment in order to properly define the motions and forces in the system. The dynamic response will be affected by the dynamic characteristics of the bridge (including the mass distribution, natural frequencies of vibration, and damping), the dynamic characteristics of the vehicle (including the speed, natural frequencies of vibration, suspension system, weight and number of axles), and the bridge surface conditions (including roadway roughness, joint discontinuities, and approach slab condition). These various parameters interact with one another, further complicating determination of the dynamic response.

Over the last 40 years, a significant amount of research has been conducted in the area of bridge dynamics, with both analytical and experimental studies being reported. However, the dynamic response is influenced by many parameters, and as a consequence, many of the studies have resulted in different conclusions. Various definitions have been used for quantifying dynamic load effects, and this has led to different conclusions being drawn from even the same set of dynamic data. Finally, the apparent dynamic response is influenced by the location and type of information being monitored, further complicating comparisons of results from different studies.

From a review of the literature, the effects of the more significant parameters on the dynamic response of bridges to vehicular loading can be summarized as follows:

Bridge Fundamental Frequency—Numerous analytical and experimental studies have shown that the natural frequency of the bridge can have a significant effect on dynamic response. If the fundamental frequency of the bridge is close to the natural frequency of a vehicle, then a state of quasi-resonance can develop and the dynamic response induced may be relatively large. The frequencies of most practical interest are in the 2 to 5 Hz range, although modes of vibration in the 10 to 15 Hz range can be excited for severe irregularities in the bridge roadway.

Bridge Span—Bridge span has historically been used by a number of codes as the sole parameter determining the magnitude of dynamic loading. While this seems reasonable, as the dynamic loading relative to the static loading will decrease with increasing span, several studies have demonstrated that the correlation of the dynamic loading fraction with span is weak at best.

Bridge Roadway Roughness and Approach Condition—Both analytical and experimental studies have shown that roughness of the surface of the bridge and the approach conditions have a significant influence on dynamic response. As roughness increases, the dynamic response also increases; this effect is amplified as the vehicle speed increases.

Bridge Type and Geometry—Structural configurations, such as simple span or continuous conditions and superstructure type, will influence dynamic response. Additionally, the geometric configuration will influence dynamic response; for instance, dynamic response will typically be higher in horizontally curved bridges and in skewed bridges when compared to straight bridges.

Vehicle Speed—Vehicle speed, combined with roadway surface conditions, will determine the vibration modes excited in the vehicle, and the crossing speed will define the duration of loading that the bridge experiences. For smooth surfaces and normally suspended vehicles, the dynamic response of the bridge is relatively unaffected by vehicle speed.

Vehicle Weight—Studies have shown that, as the weight of a crossing vehicle increases, the magnitude of the dynamic response expressed as a fraction of the static load effect decreases. The reason is that, while dynamic response increases only somewhat with increasing vehicle weight, the static response increases more rapidly.

Number of Axles—In general, as the number of axles increases, the dynamic loading factors decrease.

Number of Vehicles—Investigations have shown that the dynamic load factors associated with multiple vehicles are lower than those for single vehicles. The lower factors are due to the dynamic response from the two or more vehicles interfering with one another.

The review of design provisions showed that there exists considerable variation in the treatment of dynamic load effects. The current provisions in the AASHTO *Standard Specifications* are empirically derived and are based on experience with dynamic forces generated by steam locomotives on railway bridges in the early part of the twentieth century. Bridge dynamic loading is expressed as an impact factor that is solely a function of bridge span. Both analytical and research findings have demonstrated that dynamic response is influenced by many parameters other than span. These studies have also shown that the impact provisions in the AASHTO *Standard Specifications* are in some cases unconservative. However, in many cases the higher reported impact factors were obtained with "unusual conditions," such as severe roadway conditions or with light vehicles, and as such are not necessarily relevant for design applications.

As a result of the recent research in bridge dynamics, significant changes in procedures by a number of organizations for treating dynamic load effects have been implemented. Many codes now prescribe a "dynamic load allowance" that represents response from all types of vehicular dynamic load effects, not just impact. The AASHTO *LRFD Specifications* has adopted the use of constant factors for dynamic load allowance. This is a reasonable approach given the often conflicting effects of parameters, such as roughness, span, and configuration of vehicle. The Ontario *Highway Bridge Design Code* presented a format based on fundamental frequency of the bridge in the 1980s, but since then the format was changed to a constant factor based primarily on the number of axles controlling the design. In several of these codes, the specified dynamic load effects are somewhat larger than those in the AASHTO *Standard Specifications*.

From the survey responses, 98 percent of the U.S. agencies responding indicated that the AASHTO *Standard Specifications* alone or in parallel with the AASHTO *LRFD Specifications* are used for the design of new bridges. Only one state exclusively uses the *LRFD Specifications*. All Canadian agencies responding indicated that they are using the 1988 CAN/CSA-S6 *Design of Highway Bridges*. For overload permits, 20 states and all three provinces responding indicated that special provisions for vehicular dynamic load effects are used. Vehicle speed is restricted and the dynamic load effects considered are reduced or eliminated.

From the survey responses, 75 percent of the agencies reported there were possible field problems in existing bridges attributable to vehicular dynamic load effects. Of agencies reporting possible problems, half reported that the problems were minor. Only one agency reported wide-spread problems, with only expansion joints identified as the problem area. Approximately 70 percent of the agencies reporting problems cited expansion joints as a problem area. The other common problem areas were cracking in steel girders, connections, bearings, and concrete decks.

Field problems in some form of fatigue cracking in steel bridges were cited by approximately 70 percent of the agencies with observed problems. The use of improved expansion joint details, increased inspection, improved weld details, and the use of bolted connections wherever possible were listed as responses adopted by the agencies to the observed problems. From the survey responses, and from a review of the literature, it appears that there are no documented cases of major structural damage to bridges as a result of vehicular dynamic load effects.

The following suggestions are made for future research and for design philosophy considerations in regard to code provisions for accounting for vehicular dynamic load effects.

- Additional information could be collected from the agencies reporting field problems in existing bridges that are attributable to dynamic load effects from vehicles. This information may include detailed descriptions of the components experiencing problems, along with the solutions and/or changes in procedures adopted by the agencies. In turn, this information should be reviewed and a consensus reached for recommendations to alleviate or preclude the damage in the future.
- Many of the reported problems were either directly or indirectly associated with some form of fatigue cracking in steel bridges. The reasons for this damage could be investigated and the causes identified. For example, is the damage associated with steel fatigue in general, or is the damage specifically due to stress cycling made worse by dynamic loading effects? Are steel bridges experiencing the bulk of the reported damage because of poor detailing, lower damping, lighter superstructures, etc.?
- The term "dynamic load allowance" should be used instead of "impact" when characterizing the dynamic effects from vehicular loading, as is currently done in the *LRFD Specifications*.
- Properly accounting for dynamic effects from vehicular loading is very complex. Research has clearly demonstrated that a number of parameters have a significant influence on dynamic response, including the natural frequency of the bridge, vehicle weight and number of axles, and surface roughness. It may give a false sense of accuracy to attempt to theoretically account for only a few of the relevant parameters in determining dynamic load allowance, and the design process is likely to become too complicated if most of the significant parameters are accounted for. In fact, there is a strong need to keep the design process simple, and little justification exists in terms of improved accuracy to make the process complex. Hence, a reasonable approach is to use single or constant values for impact loading, as is currently done with the AASHTO *LRFD Specifications*. Including the number of axles when calculating effects of impact loading, as is done in the current Ontario *Highway Bridge Design Code*, may also be appropriate as little complication in the design process is introduced.

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APPENDIX A

Survey Questionnaire

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Project 20-5, Topic 28-05

DYNAMIC IMPACT FACTORS FOR BRIDGES

QUESTIONNAIRE

Name of primary respondent: ____________________

State DOT or Other Affiliation: ____________________

Title: ____________________

Phone No.: ____________________

Attached is a questionnaire seeking information on the current design and evaluation practices and observed field problems with respect to vehicular dynamic load effects.

The questionnaire is in three parts: Part 1 pertaining to the design of new bridges, Part 2 on current evaluation practices for load rating and permit loads, and Part 3 seeking information on any observed field problems. It may be appropriate for different individuals to fill out the various parts of the questionnaire. If so, please make sure the respondent for each part is identified and that the complete questionnaire is returned as a single response for the reporting agency.

Please return the completed questionnaire and any supporting documents to:

Transportation Research Board
Attn: Stephen F. Maher
NCHRP Research Synthesis
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

If you wish, you may fax your response to (202) 334-2527. If you have any questions, please call Stephen Maher at (202) 334-3245.

We would appreciate your response by May 15, 1997

THANK YOU FOR YOUR TIME AND EFFORT!!

NCHRP Synthesis Topic 28-05 Questionnaire

Agency Responding ____

PART 1 CURRENT PRACTICES FOR ADDRESSING VEHICULAR DYNAMIC LOAD EFFECTS IN THE DESIGN OF NEW BRIDGES

Name of Respondent: ____

Title: ____

Phone No.: ____

1. Does your agency use the current AASHTO *Standard Specifications for Highway Bridges* or *LRFD Bridge Design Specifications*, without modification, to account for vehicle dynamic load effects, consisting of impact factors, in the design of new bridges?

   ❑ Yes ❑ No

   If yes, indicate the AASHTO document being used:

   ❑ *Standard Specifications* ❑ LRFD *Specification*

2. If your agency is using a design methodology to account for vehicular dynamic loads which is different than that in AASHTO, please list the title of the standard in which the methodology is specified and, if possible, attach a copy of the section covering the specifications for dynamic loads.

   Design standard title ____

   Copy of specifications attached? ❑ Yes ❑ No

3. If your agency is using a design methodology to account for vehicular dynamic effects which is different than that in AASHTO, please describe when and why the different methodology was implemented ____________________

   ____

   ____

   ____

   ____

4. In your opinion, do the current procedures used by your agency for accounting for vehicular dynamic load effects result in significant complications in the design process?

   ❑ Yes ❑ No

   If yes, please explain ____

   ____

   ____

   ____

   ____

NCHRP Synthesis Topic 28-05 Questionnaire

5. Is your agency currently considering any changes to account for vehicular dynamic load effects in the design of new bridges?

❑ Yes ❑ No

If yes, please explain why and what changes are being considered ____________________

____

____

____

____

6. Has your agency previously or is it currently involved in any research studies related to the effects of or designing for vehicular dynamic loads on bridges?

❑ Yes ❑ No

If yes, please briefly describe the studies and attach copies of relevant reports ____________________

____

____

____

____

PART 2 CURRENT PRACTICES FOR ADDRESSING VEHICULAR DYNAMIC LOAD EFFECTS WHEN LOAD RATING EXISTING BRIDGES

Name of Respondent: ____________________

Title: ____________________

Phone No.: ____________________

1. How does your agency account for vehicular dynamic load effects when evaluating the load ratings for existing bridges?

❑ AASHTO *Load Rating Guide Specification*
❑ Same as for new bridge design
❑ Other

If other, please list the title of the standard in which the methodology is specified and, if possible, attach a copy of the section covering the specifications.

Design standard title: ____________________

Copy of specifications attached? ❑ Yes ❑ No

NCHRP Synthesis Topic 28-05 Questionnaire

2. When evaluating the load ratings for existing bridges, are the provisions for vehicular dynamic load effects different for routine applications, special routes, permit loads or other special cases?

❑ Yes ❑ No

If yes, please provide an explanation and attach a copy of the different provisions and describe their applications

Copy of specifications attached? ❑ Yes ❑ No

3. Is your agency currently considering any changes to account for vehicular dynamic load effects when evaluating the load rating for existing bridges?

❑ Yes ❑ No

If yes, please explain why and what changes are being considered

PART 3 FIELD PROBLEMS IN EXISTING BRIDGES ASSOCIATED WITH VEHICULAR DYNAMIC LOAD EFFECTS

Name of Respondent: ______________________

Title: ______________________

Phone No.: ______________

1. Has your agency experienced any field problems in existing bridges that are attributable to vehicular dynamic load effects?

❑ Yes ❑ Possibly ❑ No

2. If you answered yes or possibly, estimate the extent of the field problems associated with vehicular dynamic load effects within the bridges in your inventory.

❑ Minor ❑ Moderate ❑ Wide-spread

NCHRP Synthesis Topic 28-05 Questionnaire

If the information is available, estimate the number and/or percentage of bridges in your inventory that have experienced field problems associated with vehicular dynamic load effects.

Number of bridges with problems is _________ representing _________% of bridge inventory.

3. If you answered yes or possibly, were observed problems present in the main bridge structural components (e.g., main girder) or in auxiliary elements (e.g., expansion joints)?

- ❑ Main Girders
- ❑ Bearings
- ❑ Connections
- ❑ Expansion joints
- ❑ Other ______________________________

4. If you answered yes or possibly, are there common field problems and/or specific bridge types or arrangements associated with problems from dynamic load effects?

- ❑ Yes
- ❑ No

If yes, please list or explain ______________________________

5. If you answered yes or possibly, were the majority of the observed field problems associated with serviceability issues or did the problems result in failures?

- ❑ Serviceability Problems
- ❑ Failures

6. If you answered yes or possibly, have the observed field problems resulted in any revisions to the inspection, maintenance or design procedures?

- ❑ Yes
- ❑ No

If yes, please describe the changes ______________________________

NCHRP Synthesis Topic 28-05 Questionnaire

7. If applicable, please describe the observed field problems associated with vehicular dynamic load effects. Identify the type and extent of the problems. For each type of problem, include a description of the bridge type and length, characteristics of the bridge approaches, deck surface conditions, support type, age and use of bridge, expansion joint details, and any other pertinent information. Please attach additional sheets as necessary. Also, please attach any relevant reports that document the observed problems.